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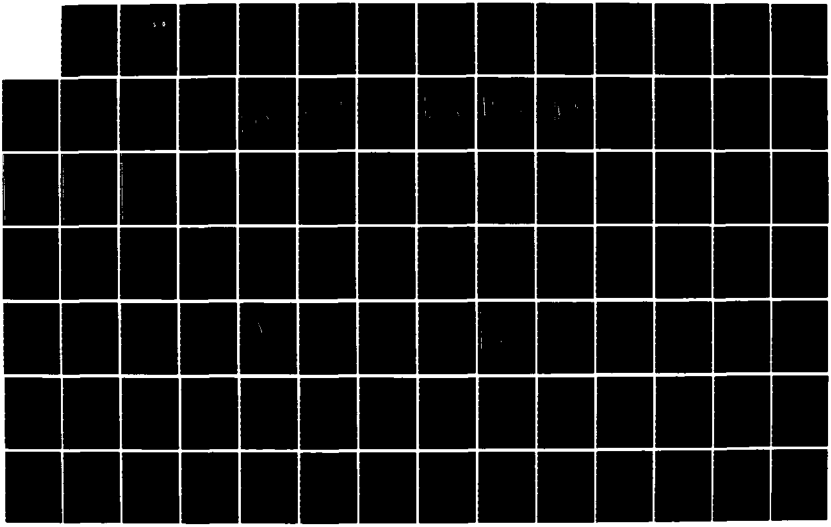

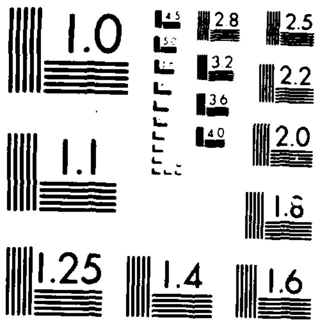

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POL PIER
FIELD INVESTIGATIONS AND RECOMMENDATIONS

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In response to requests from Military Airlift Command Headquarters, results of a field investigation of the POL pier structure at Lajes Field, Azores are reported. An engineering analysis is generated to support modification & protection recommendations including repair of fender system, construction (Con't)

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From: Commanding Officer, Chesapeake Division, Naval Facilities Engineering Command
To: Commanding General, Headquarters
Military Airlift Command (DEMU)
Scott Air Force Base, Illinois 62225

Subj: Structural Investigation of POL Pier, Lajes Field, Azores

Ref: (a) HQ MAC SCOTT AFB 132030Z JAN 78

Encl: (1) Lajes Field, Azores POL Pier Field Investigations and Recommendations, Report No. FPO-1-78(6) of March 1978

1. In accordance with reference (a), an engineering field investigation of the Lajes Field POL Pier was conducted by CHESNAVFACENGCOM personnel. Enclosure (1) summarizes the results of the investigation, provides a technical analysis of the structure, and recommends two concepts for upgrading the present facility.

2. As shown in enclosure (1) the POL Pier at Lajes Field is under-designed by present Naval Facilities Engineering Command (NAVFAC) design standards. This is supported by the history of deterioration and damage experienced by the structure.

3. While a satisfactory fender system can be designed to resist heaving abrasion and minor impact loads, the existing pier system cannot support the loads transferred to it by a T-2 class tanker (half-loaded) approaching at reasonable contact velocities with the fenders. Therefore, two alternate concepts for upgrading the facility are proposed: (a) construction of three new mooring dolphins, with fenders, to permit offloading T-5 class tankers at the present pier; repair of the deck and pile caps of the pier to reduce further deterioration; or (b) installation of a single bouy mooring (SBM) system, available as excess Navy assets, with a submarine pipeline to shore. An environmental site survey to determine suitability of seafloor for drag anchors or drilled-in-grouted pile-anchors and define sea surface environmental design criteria would be required as a prelude to selection of the SBM option.

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Subj: Structural Investigation of POL Pier, Lajes Field, Azores

4. Although firm cost estimates are unavailable with this investigation, it appears that even with the basic SBM system available as excess, the cost of an installed SBM system would be approximately three times that of the new dolphin and fender system. Other factors must also be considered such as periodic dredging, harbor pilot fees, and operational commitments for fixed or removable facilities.

5. This command is available to provide further consultation, design, site surveys, or construction support on a reimbursable basis. Discussion of subsequent efforts or questions may be directed to LT J. C. Wright, (Code FPO-1CP) at Autovon 288-3881.

G. D. CULLISON
By direction

Copy to:
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LAJES FIELD, AZORES
POL PIER
FIELD INVESTIGATIONS AND RECOMMENDATIONS

C. CHERN
LT J. C. WRIGHT

MARCH 1978

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
WASHINGTON NAVY YARD
UNDER JOB ORDER NO. 946997

ABSTRACT

In response to requests from Military Airlift Command Headquarters, results of a field investigation of the POL pier structure at Lajes Field, Azores are reported. An engineering analysis is generated to support modification and protection recommendations including repair of fender system, construction of additional mooring dolphins, or installation of a single buoy mooring system.

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TABLES

FIGURES

EXECUTIVE SUMMARY

This report is in response to the tasking of the Military Airlift Command Headquarters (MACHQ) for engineering assistance to the Civil Engineering Squadron (CES) on the POL pier at Lajes Field, Terceira Island, Azores. The task includes:

- A field investigation of the POL pier structure,
- A technical evaluation and analysis of the existing situation,
- A report of recommendations for modification of the existing POL pier structure to accommodate up to 40,000 DWT tankers.

LT J.C. Wright and Dr. C. Chern of the Ocean Engineering and Construction Project Office (CODE FPO-1), Chesapeake Division, Naval Facilities Engineering Command (CHESNAVFACENGCOM), carried out the field investigation between 31 January to 3 February 1978. A briefing of the results of the field investigation was given at the Command level on 3 February 1978 at Lajes Field. The findings on the POL pier structure were:

- The original fender system has deteriorated due to operational damage and biological attack. Repair and maintenance has been carried out frequently and at considerable cost.
- The spalling concrete around the pile caps and cracks along the center line of the concrete deck at the south bent of the POL loading platform were caused by forces in excess of the strength of the structural components.

A new fender system was proposed with the objective of transferring the berthing energy and the impact force from the mooring ships to the loading platform. The fender system will consist of wooden fender piles, continuous inner wales and chocks, fender boards and cylindrical rubber cushions. The system will be fixed at the bottom to the seafloor and at the top to the loading platform.

The theoretical analysis of the ultimate strength of the existing POL loading platform structure was performed. The results of the analysis revealed that:

- The platform structure does not possess sufficient lateral load resistance capacity to berth 40,000 DWT tankers under normal operating conditions.

- The pile cap pull-out mechanism is the first stage failure mechanism of the platform structure under a lateral force at the concrete deck level. The pile cap pull-out mechanism will not of itself cause the structure to collapse, but will induce a subsequent failure mechanism to the structure.

- The tensile yielding of the top reinforcing bars of the effective concrete T-beam section will occur following the pile cap pull-out action. The continuous yielding of the top reinforcing bars is the second stage failure mechanism of the structure. It will cause the concrete deck to crack along the center line of the platform and eventually induce the collapse of the structure. The lateral force which causes the tensile yielding of the concrete T-beam is thus the ultimate strength of the structure.

In view of the structural strength of the existing loading platform in conjunction with the persisting sediment accumulation in the Praia Bay harbor, two alternative approaches are discussed. One of the alternatives is the loading dolphin system which will require the construction of three dolphins to divert the ship impact loads from the pier structure. This plan will, however, inherit the harbor silting problem as it has been.

The other alternative is the installation of a single buoy mooring (SBM) system outside the breakwater. The SBM system will consist of a circular buoy, submarine pipe line, underbuoy and floating hoses. The advantages of this system are:

- A deep sea terminal for many sizes of tankers

- A flexible system which can be quickly removed and relocated

- Reduces or eliminates the harbor entry pilotage and tug assistance costs

- Eliminates the dredging inside the existing harbor.

Finally, the following actions are recommended:

(a) Fender System

- Detailed design of the new fender system should be initiated. The system shall be designed for T-2 class vessels.

(b) Loading Platform Structure

- Epoxy injection or cement grouting to the concrete spalling around the pile caps and cracks in the concrete deck should be performed

- A feasibility study should be initiated to consider the future operational and economical requirements of the POL system. The study shall include, but not be limited to the following approaches:

  --modification of the existing loading platform structure

  --construction of a loading dolphin system

  --installation of a SBM system

CHAPTER 1. INTRODUCTION

1.1 General Statement

The POL Pier Structure described hereinafter was constructed in early 1963 on the northern edge of the Praia Bay, Terceira Island, Azores, Portugal. Praia Bay is located in the eastern seaward side of the island as shown in figure 1-1. The bay is a crescent shape surrounded by a clean sand beach along its shore line. A breakwater extending southward was also constructed on the eastern side of the pier to protect the POL facilities from direct exposure to the open sea. Figure 1-2 is the bird's-eye view of the existing POL pier. The loading platform, shown in the center portion of the pier, is 200 feet long by 40 feet wide supported by 16" diameter steel pilings. The main function of the pier is to transfer JP-4 and diesel fuel.

According to the documented records, the pier has deteriorated since its original construction (see Appendix A of this report). The deterioration of the pier resulted in the spalling of the concrete around the pile caps and cracking of the concrete deck of the loading platform. Figures 1-3 to 1-5 illustrate typical damages of the pier structural components.

Deterioration of the pier also includes general biological attack and operational damage to the protective fender system. Figures 1-6 and 1-7 show conditions of the fender system in 1973 and late 1977, respectively. A substantial improvement in the appearance of the fender system has been achieved. However, repair and maintenance of the fender system bas become a continuous requirement.

Compounded by the high maintenance costs, inordinate consumption of manpower at the Civil Engineering Squadron (CES) and the possibility of major damage occurring to the pier structure or to ships, the Commanding General at Lajes Field requested the Military Airlift Command Headquarters (MACHQ) for engineering assistance.

1.2 Tasking

The Ocean Engineering and Construction Project Office (Code FPO-1), Chesapeake Division, Naval Facilities Engineering Command (CHESNAVFACENGCOM) was tasked by MACHQ to carry out the engineering assistance to the CES at Lajes Field. The missions of the tasking were as follows:

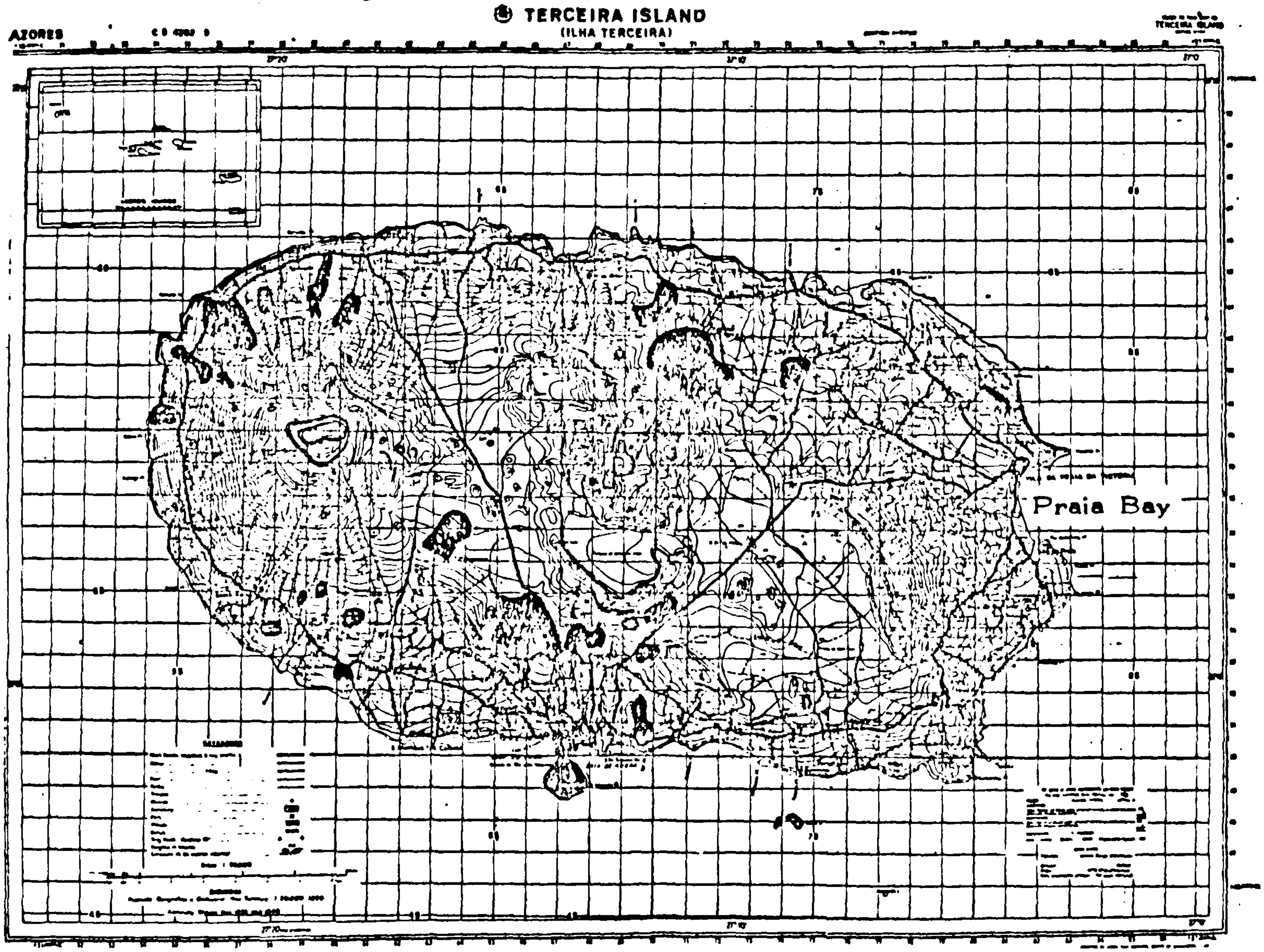

Figure 1-1 Terceira Island, Azores

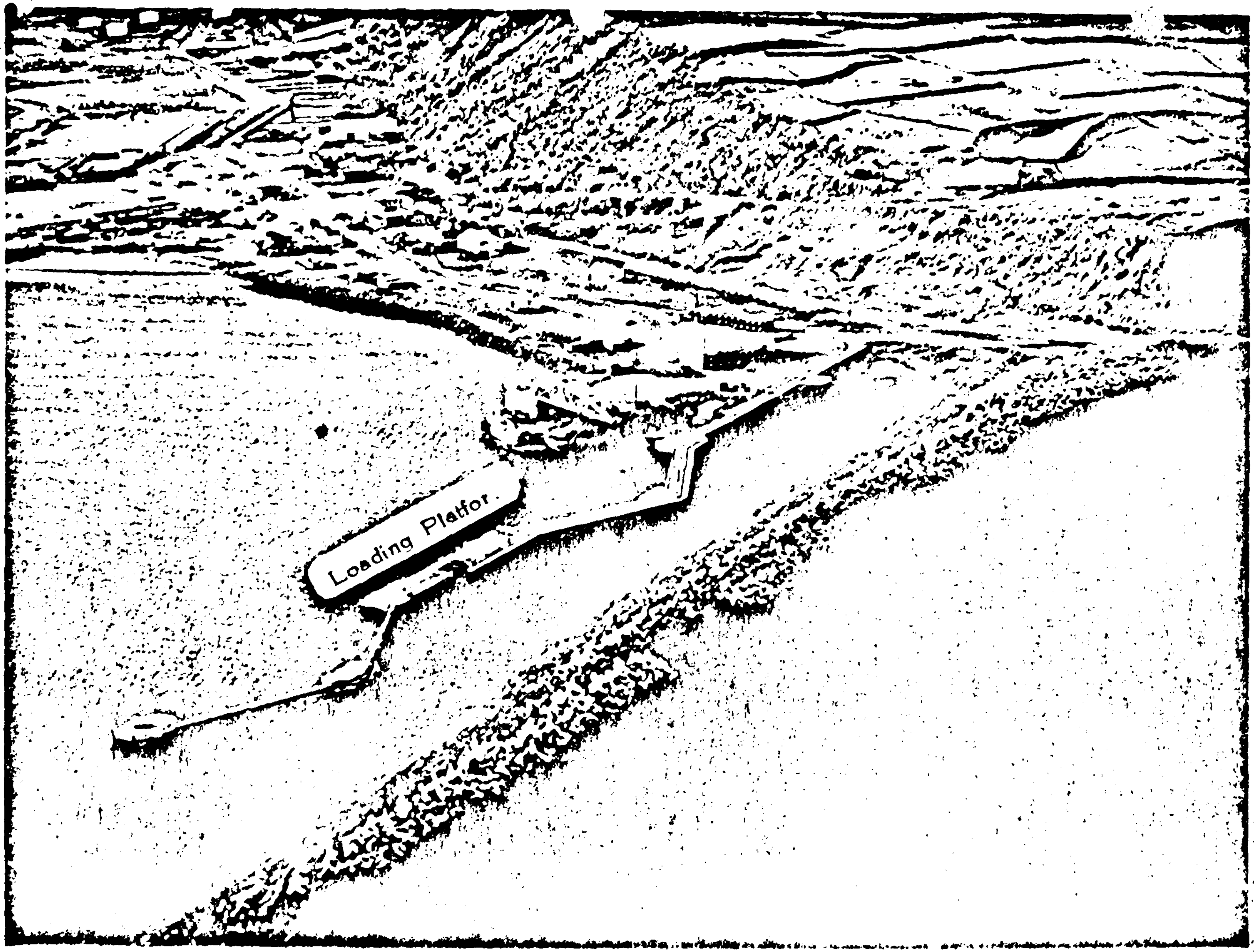

Figure 1-2 Existing POL Pier

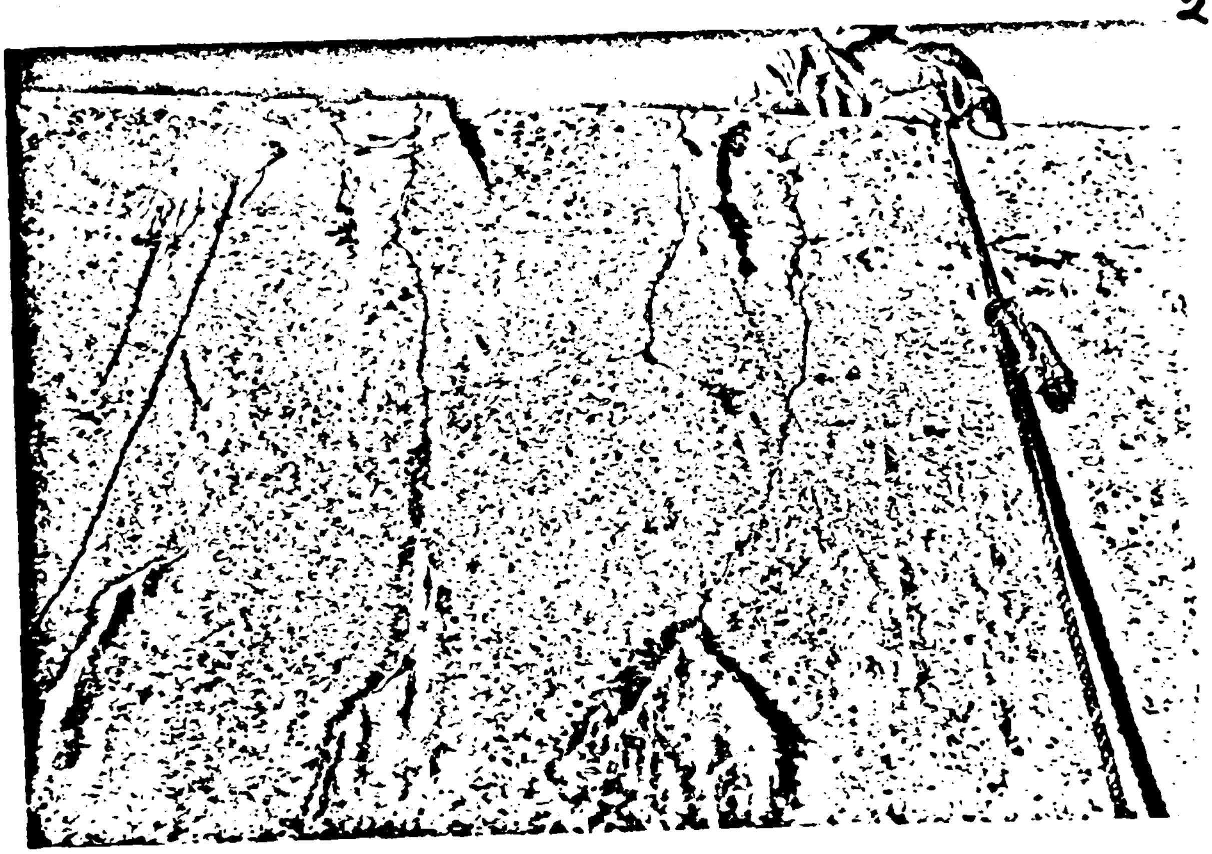

Figure 1-3 Cracks in South End of Loading Platform

Figure 1-4 Cracks in Concrete Deck at South End of Loading Platform

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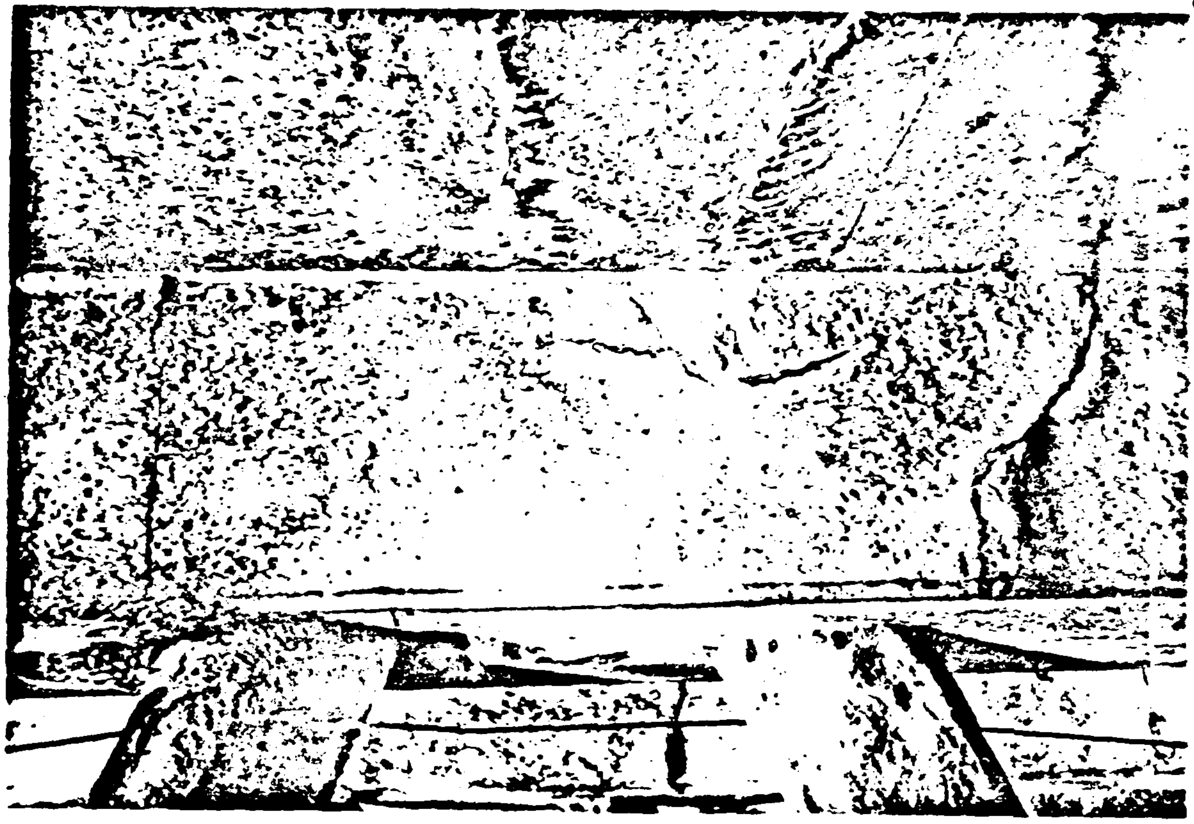


Figure 1-5 Concrete Spalling at Pile Cap and Cracks in Frame Beam

Figure 1-6 Fender System in 1973

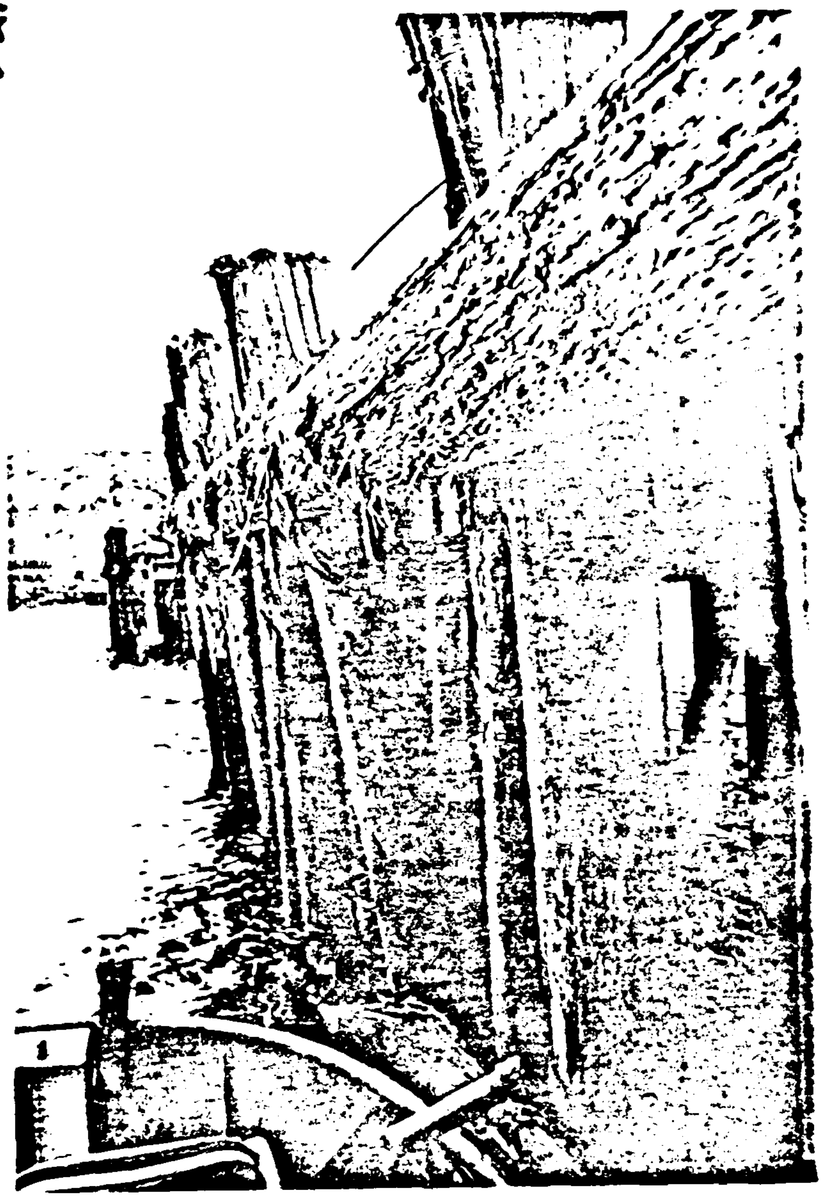


Figure 1-7 Fender System in December 1977

- An on-site visit by a marine structural engineer

- A technical evaluation and analysis of the existing situation

- A report of recommendations showing what structural modifications must be made to the pier in order to accommodate POL and cargo ships of the 40,000 ton size.

The tasking message and the financial support information to this mission are included in Appendix A of this report.

1.3 Field Investigation

In responding to the MACHQ tasking, an FPO-1 engineering team, consisting of LT J.C. Wright and Dr. C. Chern, visited the POL pier site during the period of January 31 to February 3, 1978. The objectives of the team visit were to:

- obtain design and operational data of the pier structure;

- inspect the damages of the structural components;

- inspect the existing fendering system; and

- diving inspection of the pilings.

The list of the pier structure drawings is shown in Appendix B of this report.

1.4 Findings and Possible Solutions

The findings of the POL pier inspection are as follows:

(a) Spalling of the pile caps and cracking of the concrete deck on the south bent of the loading platform were due to lateral loads applied in excess of the component strengths.

(b) Original fender system has deteriorated. The pile components are replaced at highly uneconomical frequencies.

(c) The POL pier appears to be an excellent vertical load carrying structure, similar in design to a bridge. However, the pier does not possess sufficient bracing to resist lateral load induced by ship berthing motion.

The possible solution to item (a) above is to apply cement or epoxy grouting to the cracks. Pressure grouting will not increase the strength of the structure but will protect the exposed reinforcing bars and the steel pile surfaces from environmental corrosion.

A new fender system consisted of fender piles with cylindrical rubber fenders is suggested for the possible solution to item (b) above. It is noted that the principle function of the fender system is to prevent minor impact and heaving abrasion damage to the ship and/or the pier during mooring. The fender system will be designed only to transfer the forces from the berthing ship to the pier structure. Hence, the new fender system will not reinforce or increase the strength of the pier.

The possible solutions to item (c) above constitute the major portion of this report. The suggested plans are:

- Loading Dolphin Approach - The plan calls for the construction of three loading dolphins in front of the loading platform to resist the impact force from berthing ships. Thus, the pier structure will be free from direct impact of larger size ships.

- SBM System Approach - The plan calls for the installation of a single buoy mooring (SBM) system outside the breakwater. The ships will moor to the SBM and then transfer the liquid fuel shoreward through submarine pipe lines. In this system, the oil tanker will not use the existing POL pier.

The briefing at the Command level at Lajes Field following the investigation is documented in Appendix C of this report.

CHAPTER 2. EXISTING POL PIER AND ENVIRONMENT

2.1 Introduction

The POL pier consists of a loading platform, four mooring dolphins, traffic bridges, and walkways. Figure 2-1 illustrates the general arrangement of the pier system. The bridges to the north of the loading platform are 12 feet wide and are designed for HS20-44 truck loading in accordance with AASHO specification (See Appendix B.1 File No. 7571-5782 of this report). The walkways to the south of the loading platform are 5 feet wide and are designed to carry 100 psf uniform live loads. The supports to the bridges and walkways are all 12 3/4" Ø x 1/2" WT steel piles penetrating approximately 40 feet into the seafloor.

Mooring dolphin No. 1 is supported by a steel sheet piling cofferdam with stone backfill and mooring dolphins Nos. 2 to 4 and the loading platform are supported by a series of 16" Ø x 1/2" WT steel pipe pilings. Both the dolphins and loading platform are designed for T-2 class tanker moorings at 65 MPH wind conditions.

Fixed wooden pile fender systems are attached to the loading platform and mooring dolphin No. 4. The original fender system has deteriorated and the replacement of wood fender pilings has become a continuous operation.

2.2 Subsea Soil Data

Subsea soil data in the vicinity of the POL pier site are available (See Appendix B.1, File No. 7571-5783 of this report). Figures 2-2 to 2-4 depict the boring logs of sites Nos. T-3, T-5 and T-8. The approximate locations of the boring sites Nos. T-3, T-5 and T-8 are shown in figure 2-1.

The seafloor materials in the vicinity of the pier site are composed of a black volcanic sand with a high shell content which gives it a "salt and pepper" appearance. Based on the results of mechanical analyses of samples taken from boring sites, the materials along the shore and immediate off-shore area generally consist of a well-sorted sand with the mediam diameter grain size falling predominantly in the range of fine sand.

The sand layer is approximately 40 feet deep. Hard basalt lies thereunder.

2.3 POL Pier Structural Component Strength

The loading platform is the prime structure of the POL pier system. The platform consists of a concrete deck 200 feet long by 40 feet wide supported by a series of 16" Ø x 1/2" WT steel pipe pilings. The plan view of the concrete deck and the cross-section of the platform are shown in figure 2-5. The wooden fender system is attached to the shipward face of the 60-foot sections on both ends of the concrete deck.

The structural component strengths calculated in this section are the basic factors defining the ultimate strength of the loading platform under

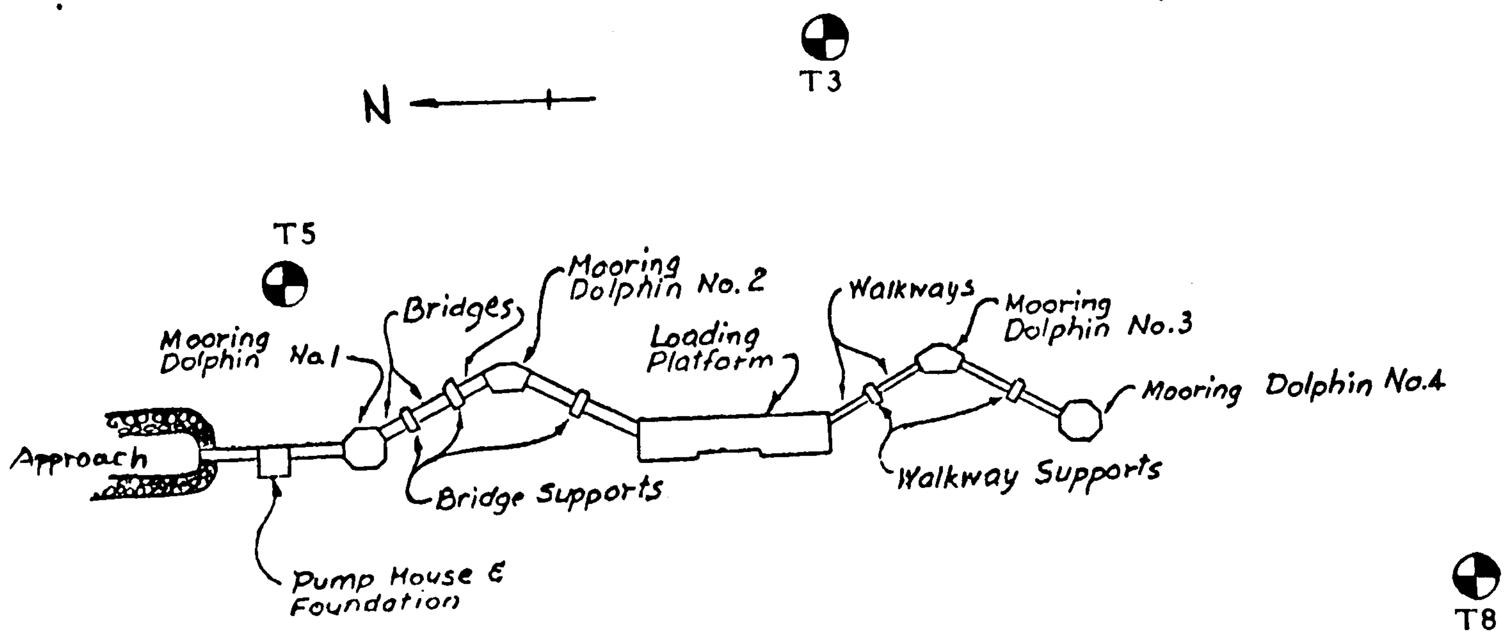


Figure 2-1 POL Pier General Arrangement

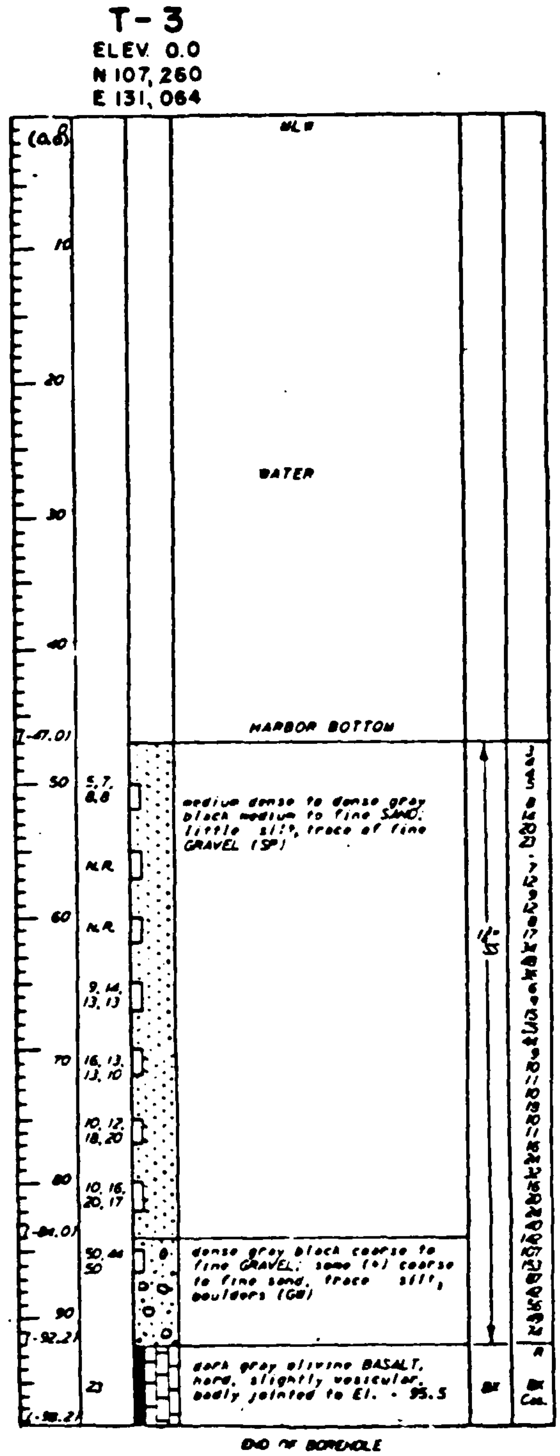


Figure 2-2 Soil Boring Log -- Site No. T-3

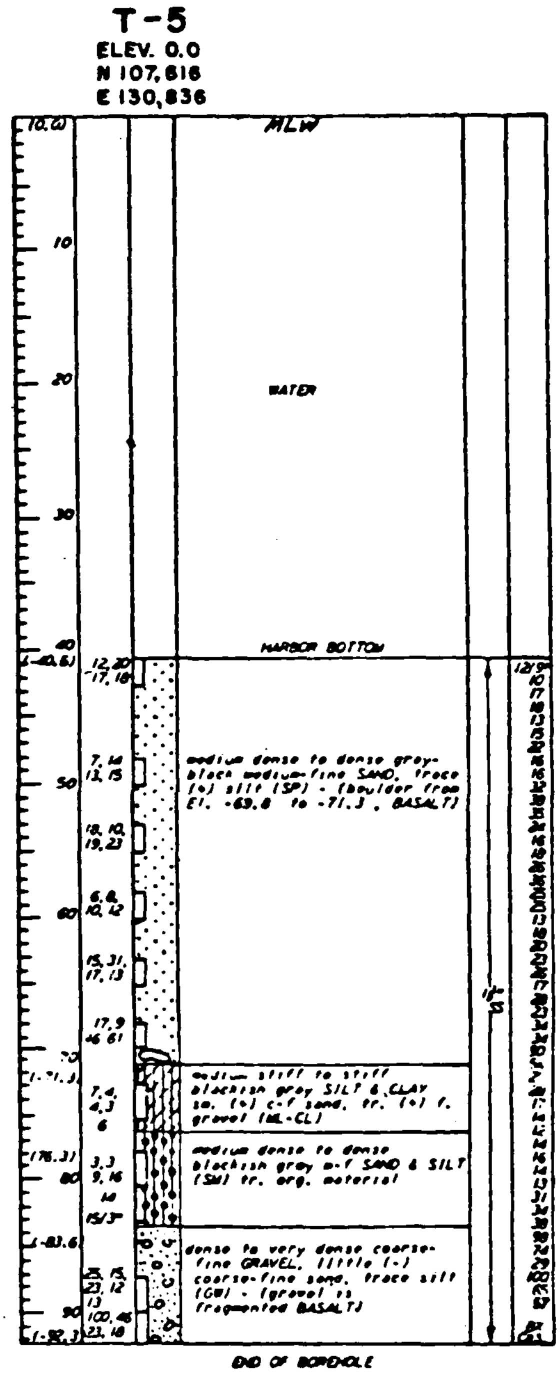


Figure 2-3 Soil Boring Log -- Site No. T-5

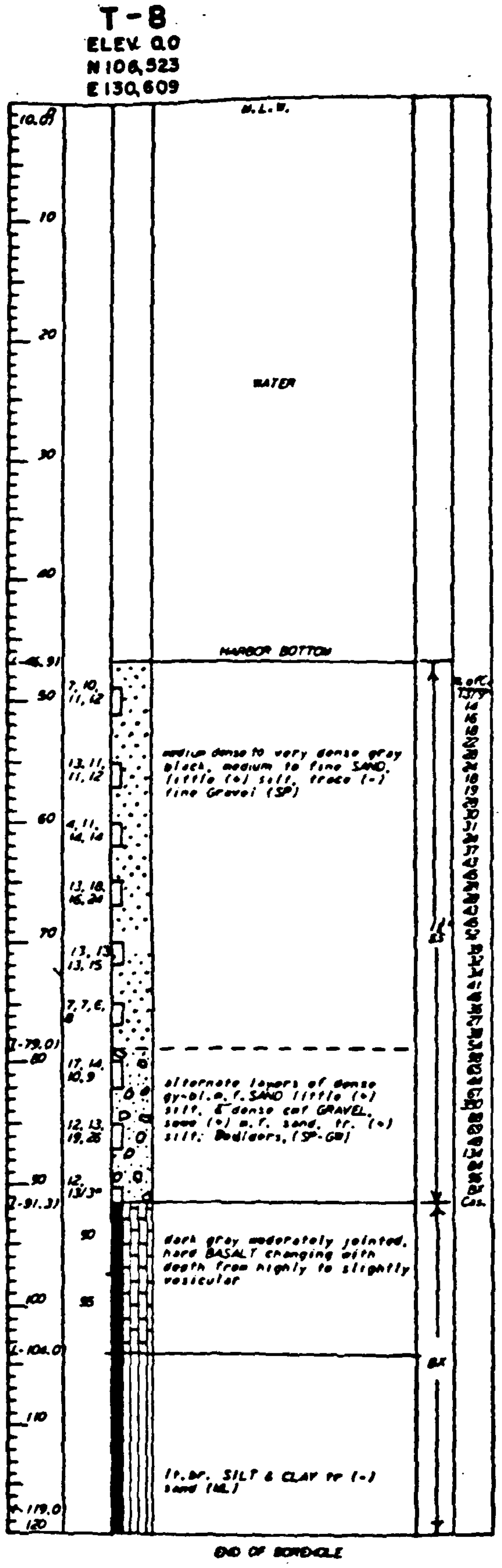


Figure 2-4 Soil Boring Log -- Site No. T-8

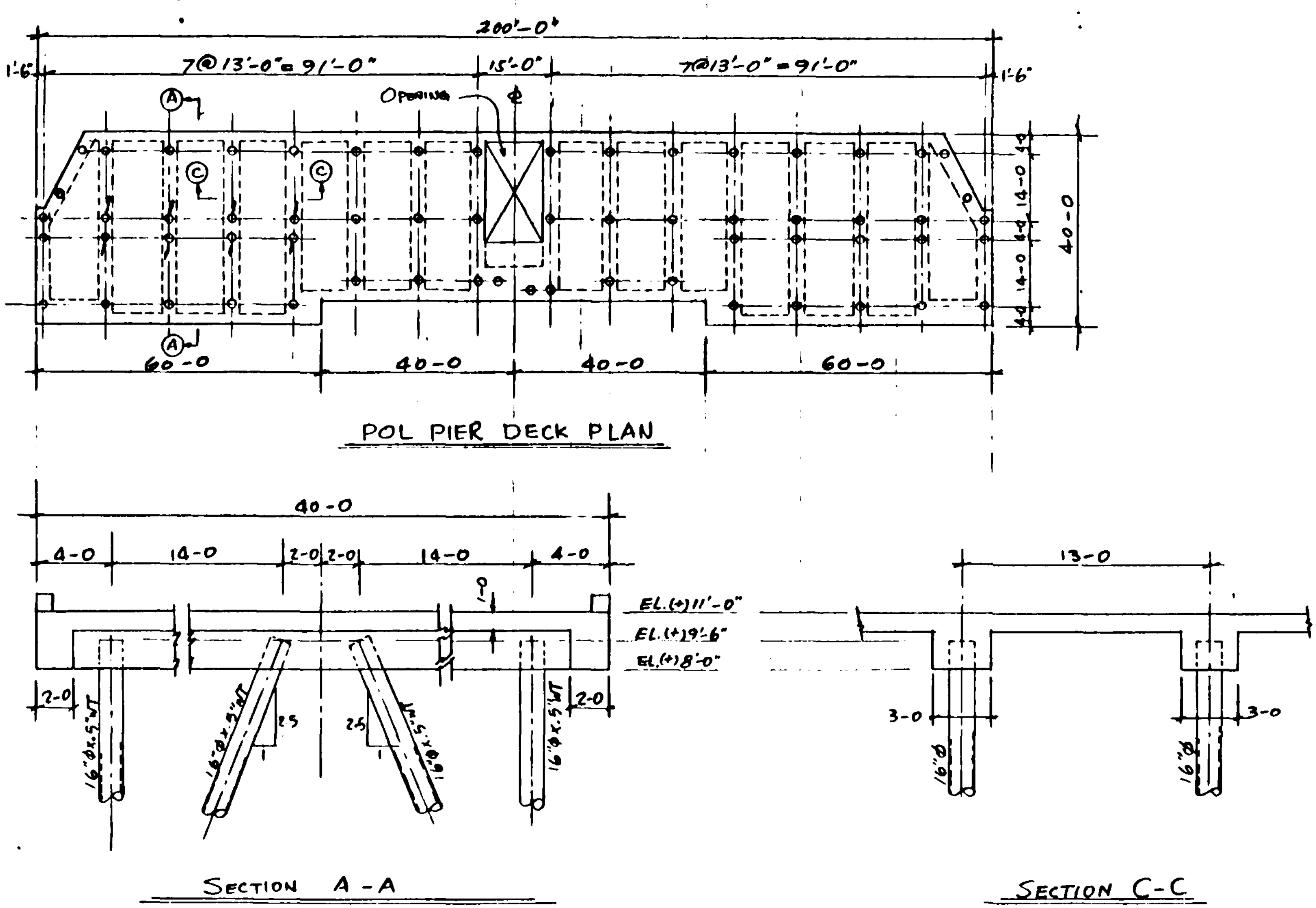


Figure 2-5 POL Pier Loading Platform

lateral loads induced by berthing ships. These basic factors are the pile cap pull-out strength, pile foundation pull-out strength and the concrete deck T-beam strength under negative moment.

2.3.1 Pile Cap Pull-Out Strength

Figure 2-6a illustrates the pile cap pull-out mechanism. The 16" Ø pile is embedded into the concrete beam a distance of 1 foot 6 inches. The equilibrium condition is:

$$Pu = \pi D \cdot \ell \cdot \mu$$

where Pu = ultimate pull-out force, lbs

D = pipe pile diameter, inches

ℓ = pipe pile embedded length, inches

μ = bond stress between concrete and steel surface, psi

In the absence of reliable data on the bond stress between concrete and steel surfaces, the allowable stress of 20 psi with a factor of safety of 2.0 is used to compute the value of Pu (See Reference 4). Appendix D.1 shows the detailed calculation which yields Pu = 36,000 pounds.

2.3.2 Pile Foundation Pull-Out Strength

A pipe pile under axial tension is illustrated in figure 2-6b. The ultimate pull out force is computed by the following expression:

$$Qu = \Sigma f_i \, (\Delta A)_i$$

where Qu = ultimate pull-out force, lbs

f_i = unit skin friction between sand and the pile steel surface at depth i, psf

ΔA_i = pile surface area at depth i, sq. ft.

Appendix D.2 shows the detailed calculation of the ultimate pull-out force Qu. It gives

Qu = 62,800 lbs.

It is noted that the pile foundation pull-out strength is greater than the value of the pile cap pull-out strength. Therefore, the pile will fail in the pile cap location prior to tensile failure in the foundation.

2.3.3 T-Beam Strength Under Negative Moment

A typical deck beam cross-section of the loading platform is shown in figure 2-6c. The T-beam consists of a concrete deck 13 feet wide by

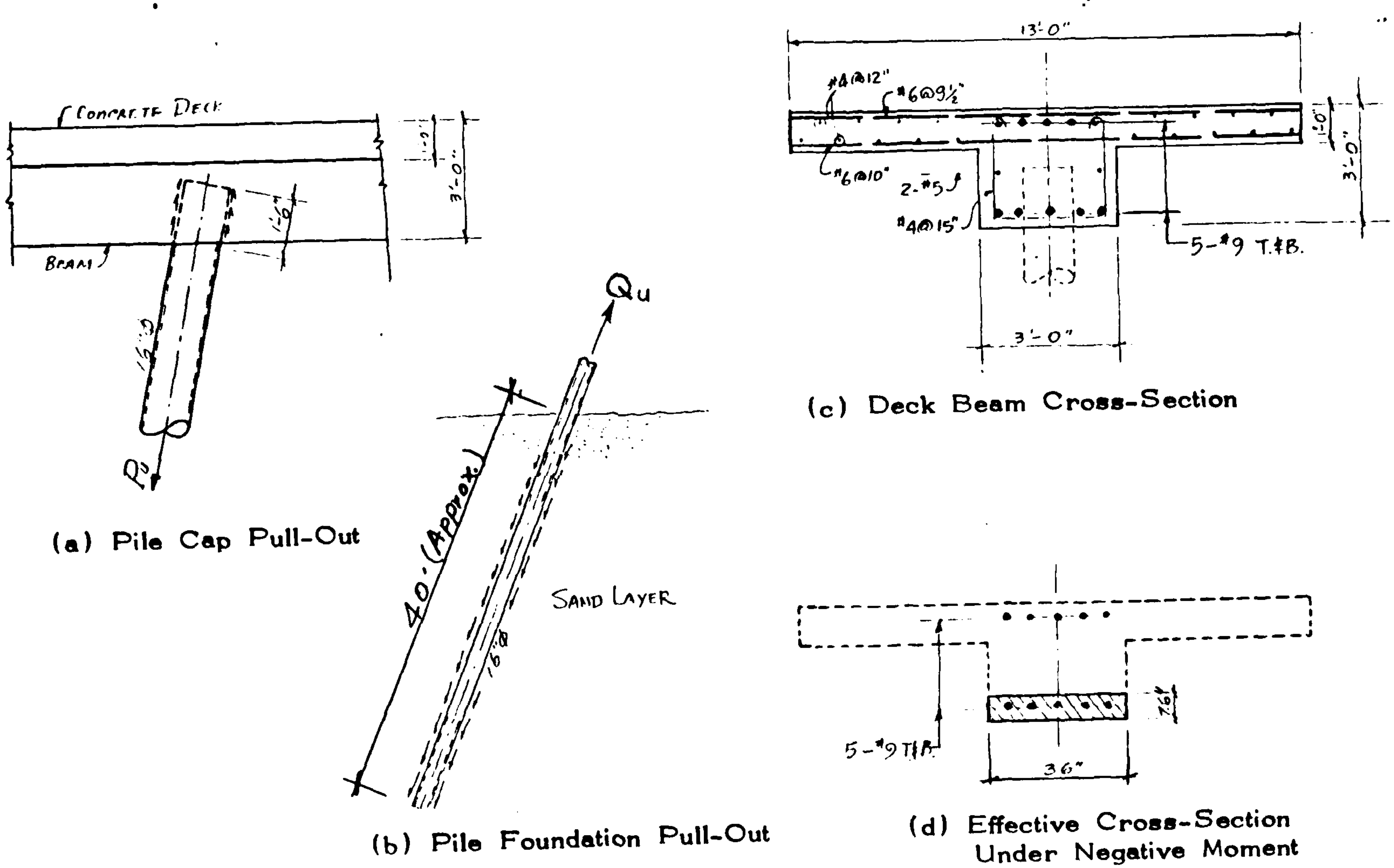


Figure 2-6 POL Pier Structural Components

1 foot thick as the flange and a 3 feet wide by 3 feet deep concrete pile cap beam as the stem. The main reinforcing steel are 5 - #9 bars in the top and bottom of the beam.

When the T-beam section is subjected to a negative moment, the top concrete fibers of the beam will be under tension. If the negative moment is greater than a certain amount, that is, the moment causing the fiber stress in excess of the concrete tensile strength, the concrete will crack and loose its load carrying capacity. Under such circumstance, the T-beam section is transformed into an effective cross-section as shown in figure 2-6d. The ultimate moment capacity of the effective cross-section will be governed by either the tensile yielding of the top reinforcing bars or the compression failure of the concrete in the bottom surface of the beam. According to the computations in Appendix D.3 the ultimate strength of the T-beam under negative moment is governed by the yielding of the top bars. The moment value is found to be

$$Mu = 424.6 \text{ ft-kips}$$

2.4 Oil Tanker Characteristics

The length, beam, draft, and the capacity or tonnage of the oil tankers that will use the off-loading facilities will have a direct bearing on the design of the approach channel, the pier or the offshore terminal. These characteristics are shown in Tables 2-1 and 2-2, for U.S. Navy oilers and representative commercial oil tankers.

Additional ship characteristics of the U.S. Navy oilers such as wind force and moment can be referred to in reference 11.

2.5 Environment

The environmental factors of winds, waves, tides and currents can be found in reference 8 and file Nos. 7571-5785 to 7571-5791 shown in Appendix B. The following abstracts are from these references.

2.5.1 Winds

The prevailing winds are southwesterly and northwesterly. Those of greatest velocity approach Terceira Island from the Southwest. Winds from the north-east through southeast create waves which enter Praia Bay and find their way to the beach. Hurricane winds have been observed with a maximum velocity of 80 MPH for a duration of 2 hours. Gusts have been recorded as high as 86 MPH.

2.5.2 Waves

Waves from the easterly direction, to which Praia Bay is exposed, seldom exceed 8 feet, and in most instances average 3 feet or less in height. During storms, however, the greatest waves come from the southeast and are generated by storms from the south. Waves of 18 feet in height with

Table 2-1 Characteristics of United States Navy Oilers (From Reference 9)

Year Built	Oiler Number	Length (ft) Water Line	Length (ft) Over-all	Breath (ft)	Draft (ft) Full Load	Displacement (Long Ton) Light Load	Displacement (Long Ton) Full Load
	T2-A Type						
1942	AO 36		501.4	68	30.75		21,580
1942	AO 43						
	T3-S-2A1						
1940	AO 25		553	75	31.5		25,525
1943	AO 54						
1943	AO 56						
1944	T-AO 57						
1944	T-AO 62						
1945	AO 64						
	T3-S3-A1						
1943	AO 51		644	75	31.5		34,750
1945	AO 98						
1945	AO 99						
	T3-S2-A3						
1945	T-AO 105		646	75	35.5	11,000	34,750
1946	AO 106						
1946	T-AO 107						
1946	AO 108						
1946	T-AO 109						

Table 2-1 (Continued)

Year Built	Oiler Number	Length (ft) Water Line	Over-all	Breath (ft)	Draft (ft) Full Load	Displacement (Long Ton) Light Load	Full Load
	"Neosho" Class						
1954	AO 143	640	655	86	35	11,600	38,000 to 40,000
1955	AO 144						
1955	AO 145						
1955	AO 146						
1955	AO 147						
1956	AO 148						
	New Construction						
FY76	AO 177		586.5	88	33.5		27,500
FY76	AO 178						
FY77	AO 179						
FY77	AO 180						
FY78 to 80	AO 181 to AO 185						
	AOE Type						
1964	AOE 1		793	107	39.3	19,200	53,600
1967	AOE 2						
1969	AOE 3						
1970	AOE 4						

Table 2-1 (Continued)

Year Built	Oiler Number	Length (ft)		Breath (ft)	Draft (ft)	Displacement (Long Ton)	
		Water Line	Over-all		Full Load	Light Load	Full Load
	AOR Type						
1969	AOR 1		659	96	33.3		38,100
1969	AOR 2						
1970	AOR 3						
1970	AOR 4						
1971	AOR 5						
1973	AOR 6						
1975	AOR 7						
	M.S.C.						
1956	Maumee		620	93.4	33.6		26,943
1957	Shoshone						
1957	Yukon						
1959	American Explorer		615	86	36		24,298

Table 2-2 Characteristics of Commercial Tankers (Data from Reference 10)

Dead Weight Tonnage (Long Tons)	Displacement Tonnage (Long Tons)	Over-all Length (FT)	Breadth (FT)	Fully Laden Draft (FT)
20,000	26,700	584	73.5	31.2
25,000	33,300	623	79.4	32.8
30,000	40,000	656	84.6	33.8
35,000	46,600	682	90.0	34.8
40,000	53,200	705	95.0	35.8
45,000	60,000	732	100.0	36.7
50,000	66,600	755	105.0	37.4
65,000	86,500	820	111.0	43.6
85,000	113,000	853	125.0	46.0
100,000	133,000	935	135.0	47.9
200,000	266,600	1,017	155.0	62.0
300,000	400,000	1,112	175.0	71.8
400,000	533,000	1,214	187.0	87.6
500,000	666,600	1,306	226.0	92.0

the period of 9 to 11 seconds were recorded during winter of 1955 at the old breakwater in Praia Bay. The duration of these 18-foot waves was approximately 12 hours. Information on storm waves indicates that wave heights at the new breakwater during the storm of December 1962 were estimated to have reached a maximum of 25 feet. The wind velocity during this storm was of 25 to 40 knots east winds.

2.5.3 Tides

Tides in Praia Bay are semidiurnal with a mean range of 3.7 feet, and a spring range of 4.9 feet. Highest high water is 5.0 feet above mean low water and lowest low water is 1.7 feet below mean low water.

Strong east winds tend to increase the tidal height in Praia Bay, while strong west winds will tend to reduce it. Information on the frequency and amplitude of storm tides are not available.

2.5.4 Currents

Current measurements were performed inside Praia Bay. The results indicate that there is a clockwise rotation of water in the northern section of the bay. The currents are relatively constant with depth and no appreciable change in direction with depth. Current velocities inside the bay vary from 0.00 to 0.80 knots with the maximum velocity to the southeast end of the bay.

2.6 Harbor Soundings

The harbor soundings in the vicinity of the POL pier are shown in figures 2-7 and 2-8 which were surveyed in 1970 and 1977, respectively. A direct comparison of the seafloor elevation at a specific location can not be obtained from these two sounding diagrams. However, an approximate 300 feet wide by 35 feet deep approach channel to the POL pier appears to be properly maintained throughout this period.

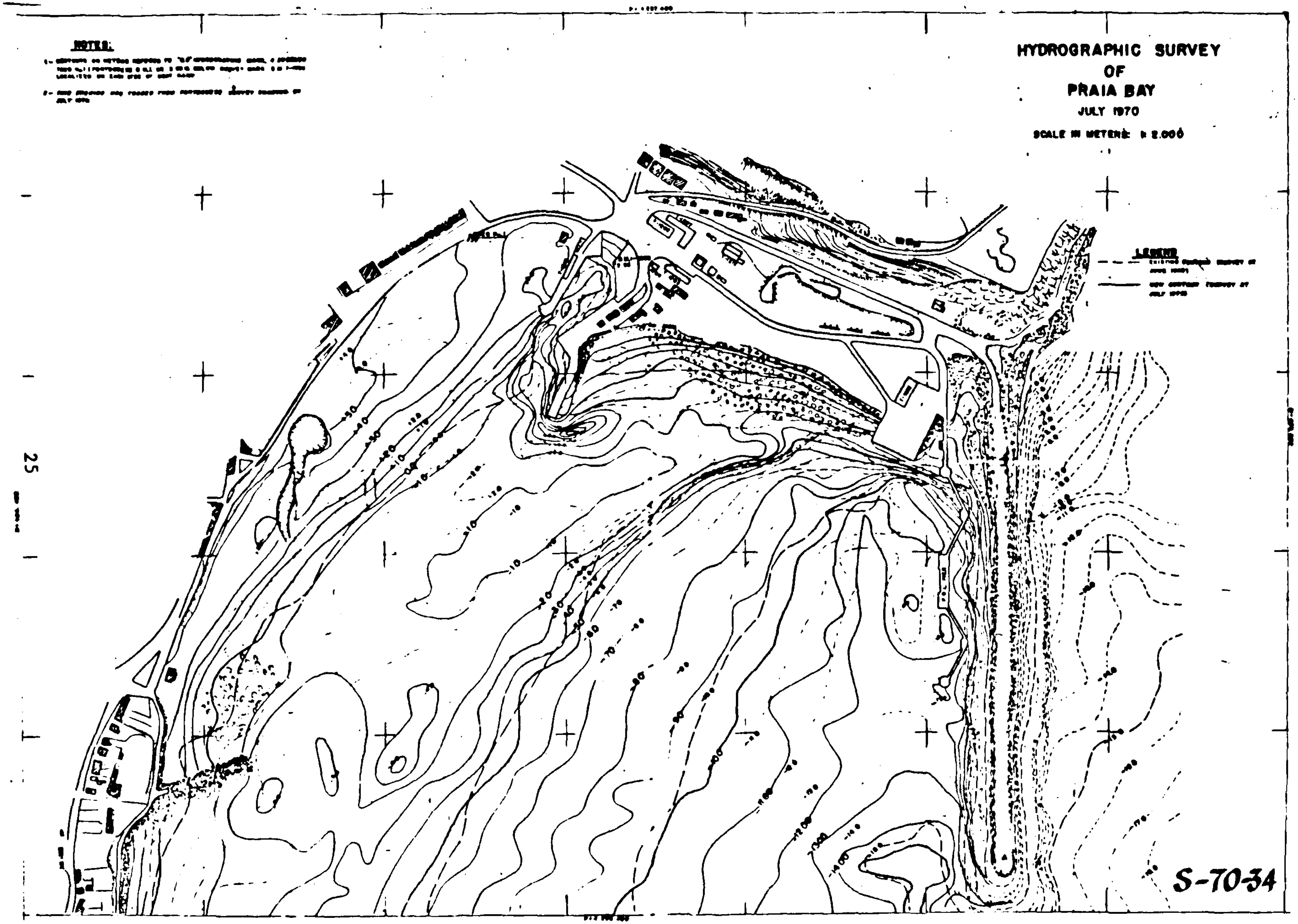


Figure 2-7 Praia Bay Harbor Sounding in 1970

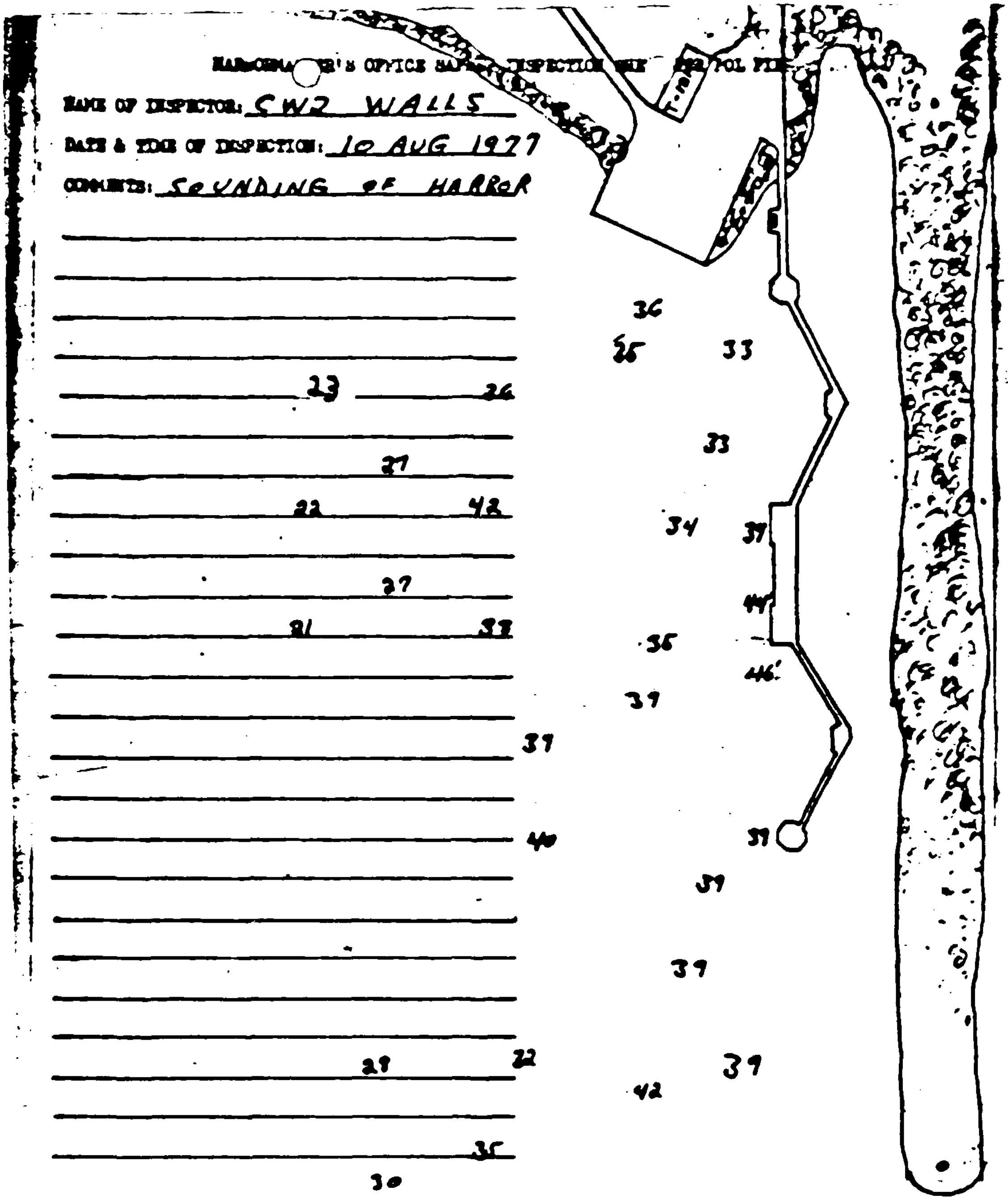


Figure 2-8 Praia Bay Harbor Sounding in 1977

CHAPTER 3. FENDER SYSTEM

3.1 Introduction

The new fender system proposed in this section will be attached to the shipward side of the two - 60 foot sections of the loading platform. The system in each side of the platform will consist of a group of twelve vertical wood pilings, two continuous horizontal wales with chocks, two fender boards and two cylindrical rubber fenders. Figure 3-1 illustrates the general arrangement of the proposed fender system, the side view and the front view of the proposed system are shown, respectively, in figures 3-2 and 3-3.

As shown in the figures, the wood pilings will be driven at least 10 feet into the seafloor. The upper end of the pilings will be fastened to the continuous horizontal wales at EL. (+) 10'-0" and EL. (+) 2'-6" measured from the mean low water, respectively. Chocks will be bolted to the wales to prevent pilings from rolling.

Two 12 feet by 12 feet fender board, shown in figure 3-1, will be fastened to the pilings to serve as reaction surfaces to the cylindrical rubber fenders.

Structural members of the proposed fender system were arranged by basing upon the material specification which follow:

(a) Wooden Fender Piles

- Class II treated southern pine or douglas fir
- Limiting flexural stress of the extreme fiber in tension is at 1,750 psi
- A nominal 12-inch diameter wood piling will have approximately 18-inch in diameter at the end section of the piling

(b) Cylindrical Rubber Fender

- SEA CUSHION 4'ØX7.4'ℓ rubber fender or equivalent

The calculations regarding to the energy absorption and impact force transferring function of the fender system are compiled in Appendix D.4 of this report. It is noted that the computed maximum lateral reactions at both ends of the pilings are limited by the flexural stress of the extreme fiber in tension. The reaction forces at the lower end of the piling were used as the base to define the pile penetration requirement.

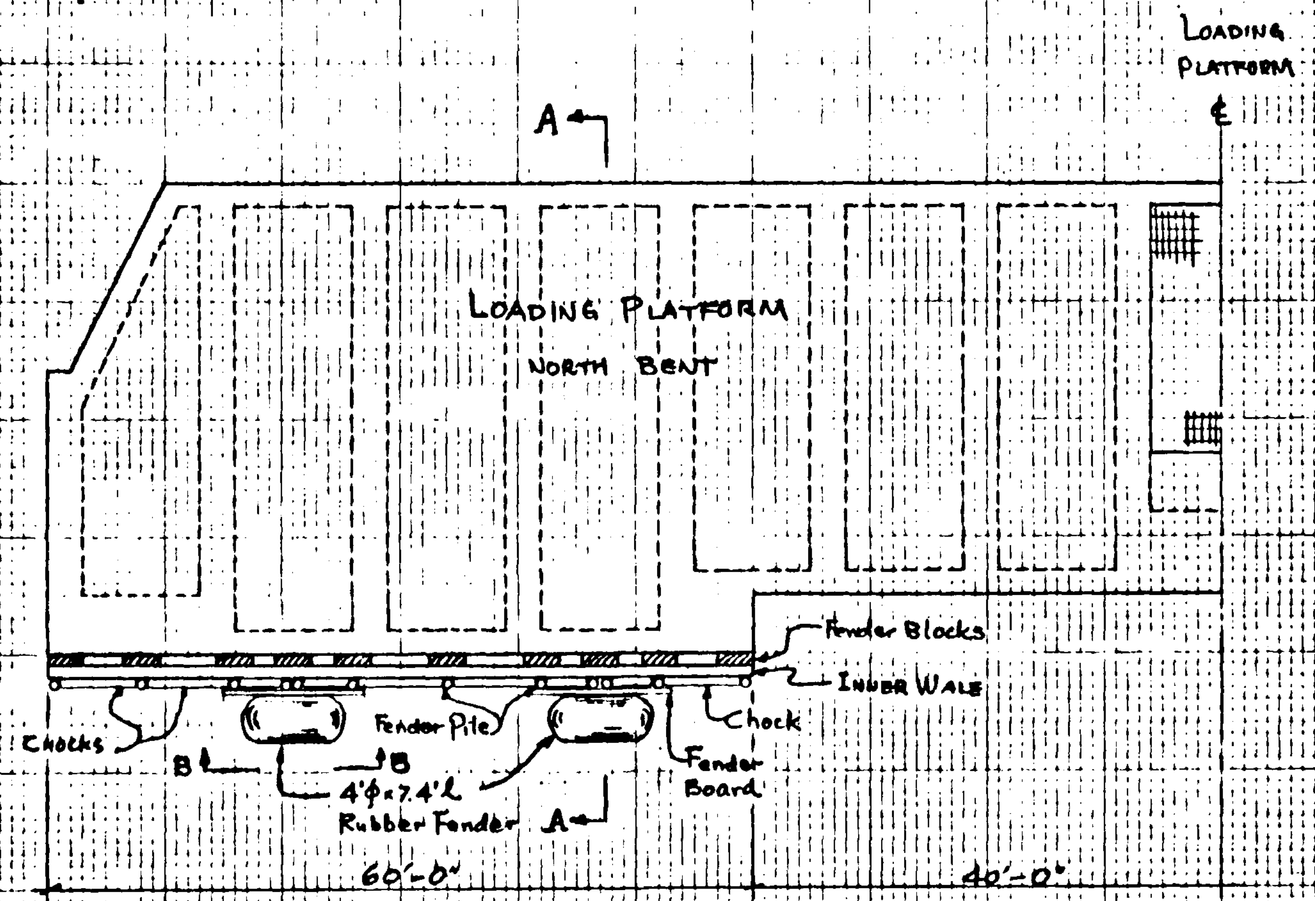


Figure 3-1 Proposed Fender System -- Plan View

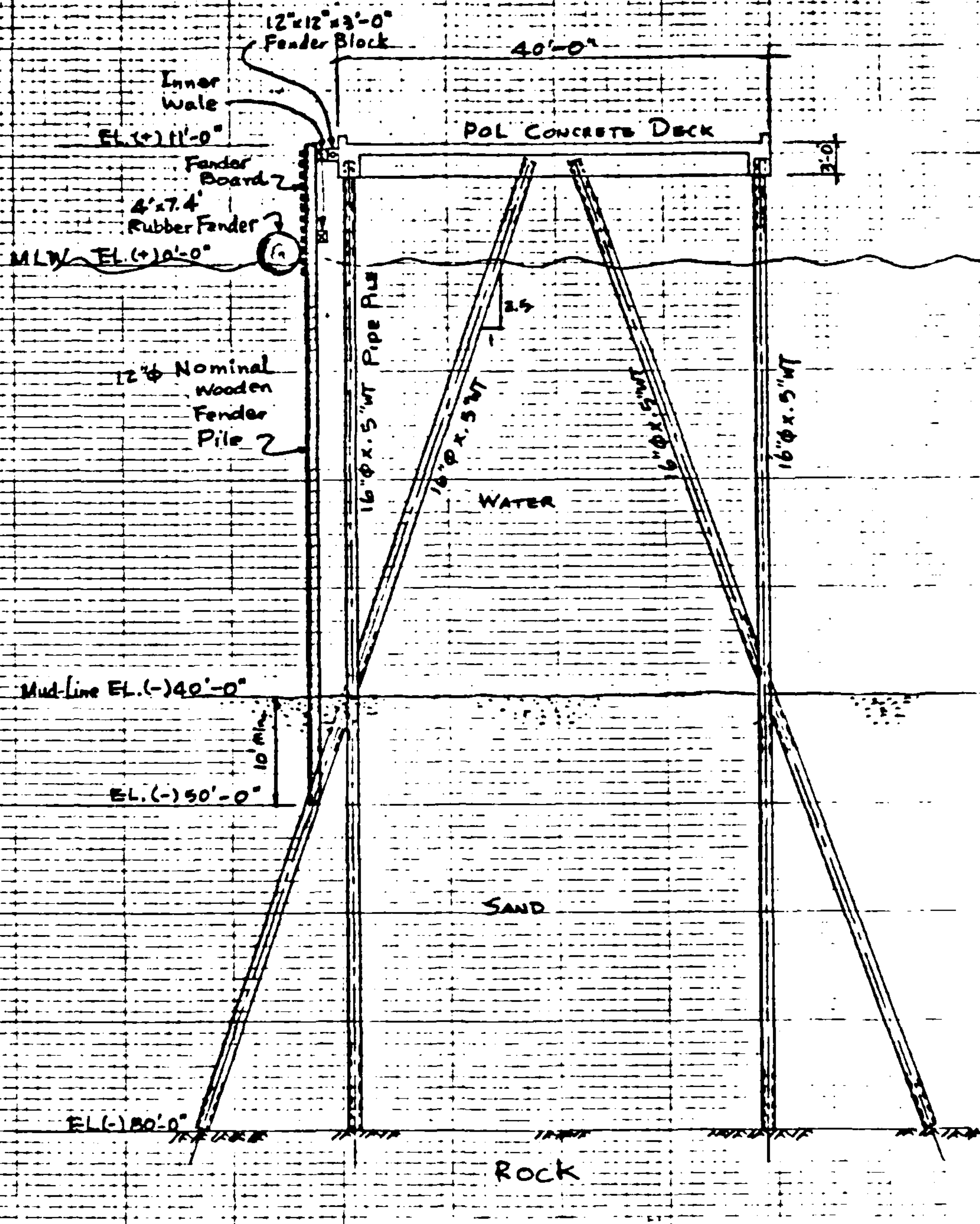


Figure 3-2 Proposed Fender System -- View A-A

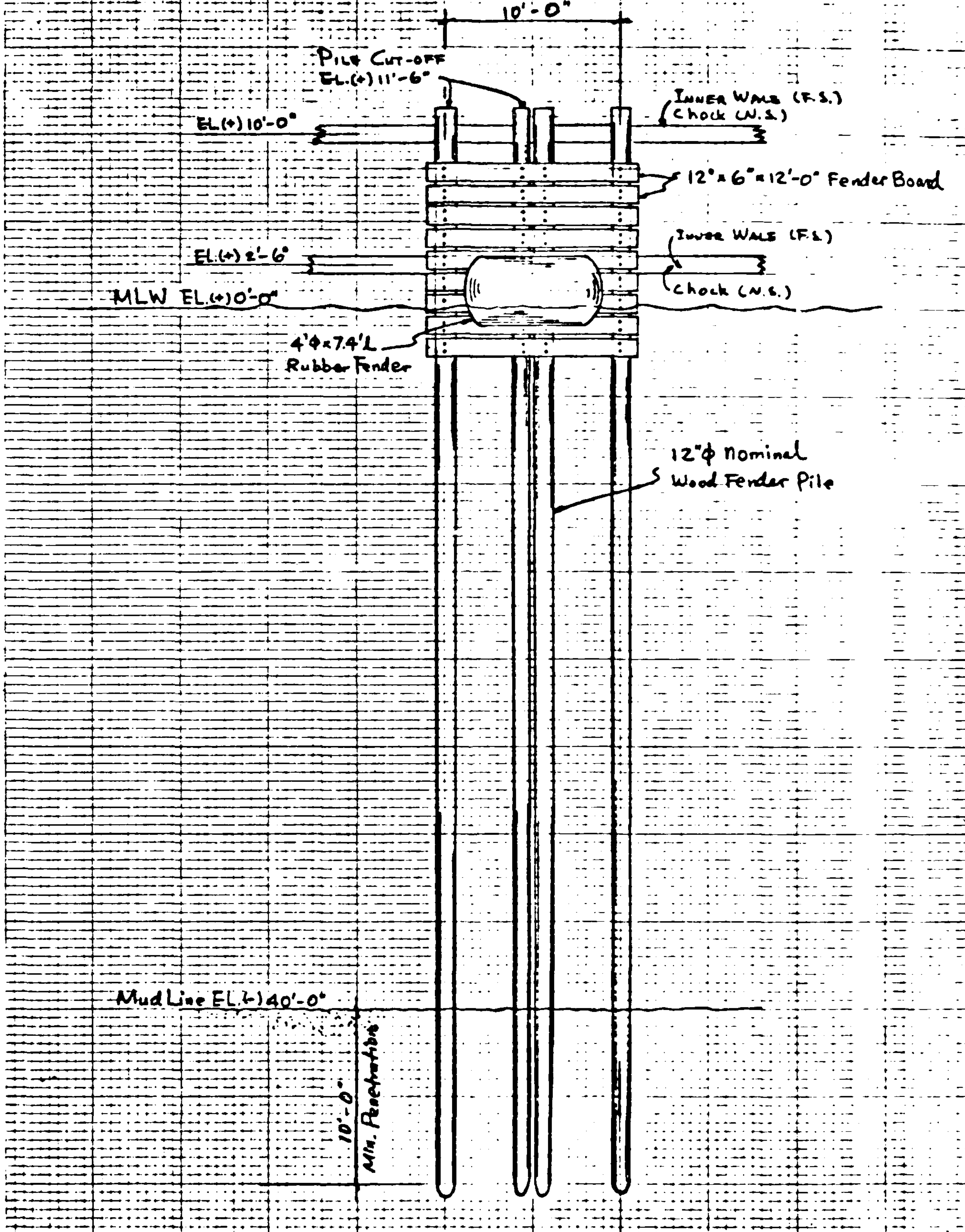


Figure 3-3 Proposed Fender System -- View B-B

3.2 Pile Penetration Requirement

Main factors determining the penetration requirement of the fender pilings are:

- pile end lateral reaction force
- seafloor scouring around the piling

The calculated maximum reaction at the lower end of the fender pile is approximately 2,200 lbs per pile. According to the design curve for 12"Ø pile under lateral loads, as shown in figure 3-4, the required penetration is 3.5 feet. The calculations of the pile lateral resistance capacity in sand is shown in the Appendix D.5.

In the absence of reliable data for determining the effect of local scouring at the POL pier site, a depth of 6 to 7 feet of scouring around the fender pilings is assumed. The total penetration requirement of the fender pilings is at about 10 feet. It is noted that a minimum of 10 feet penetration is annotated in figures 3-2 and 3-3.

3.3 Pile Driving Resistance

In order to drive the pilings to grade, the fender pile driving resistance curve was derived as shown in figure 3-5. It is noted that the axial resistance of a 12"Ø pile at 10 foot penetration in sand will be approximately 25,000 lbs.

Pile driving resistance computations are compiled in the Appendix D.6.

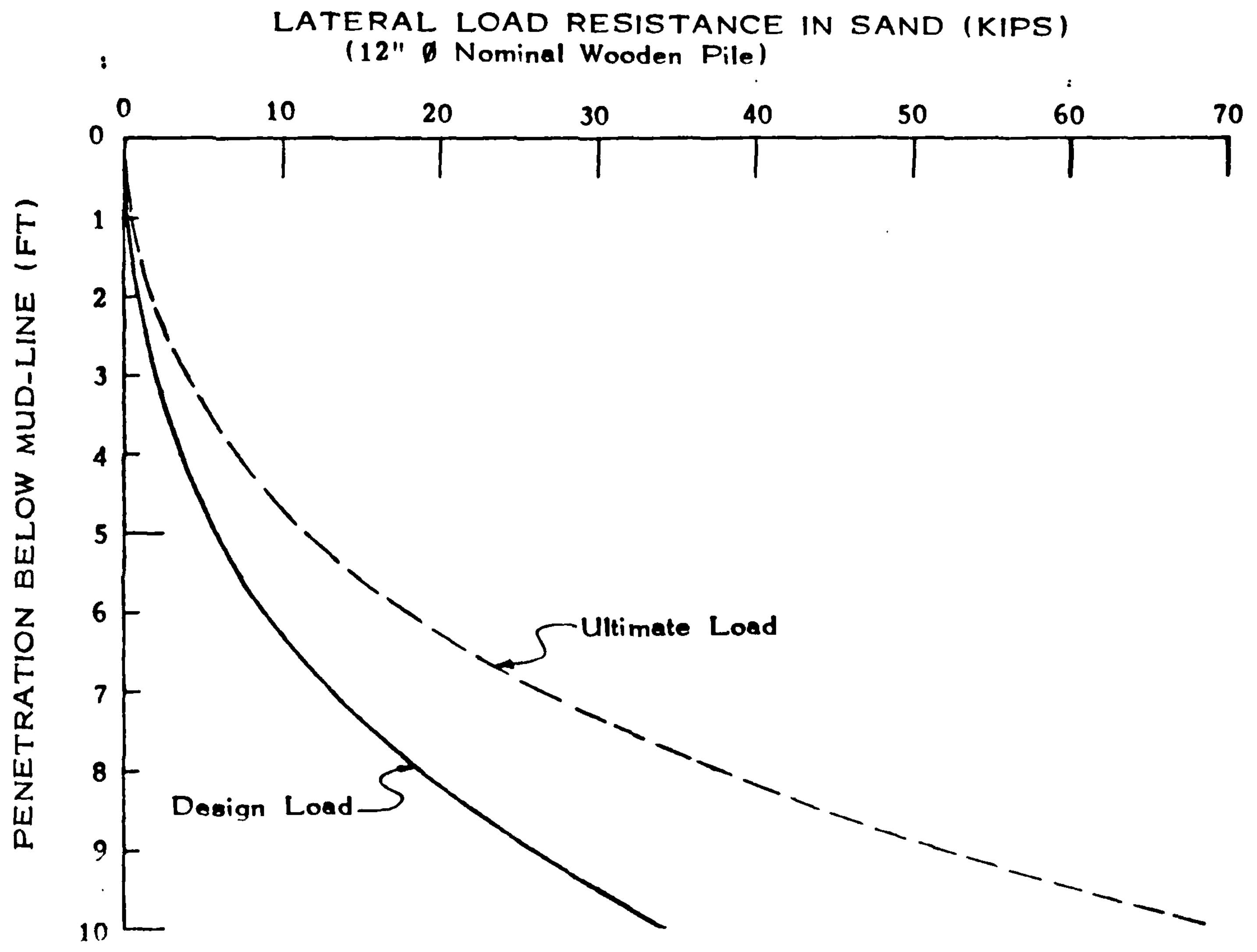


Figure 3-4 Fender Pile Lateral Load Resistance in Sand
(Data from Appendix D.5)

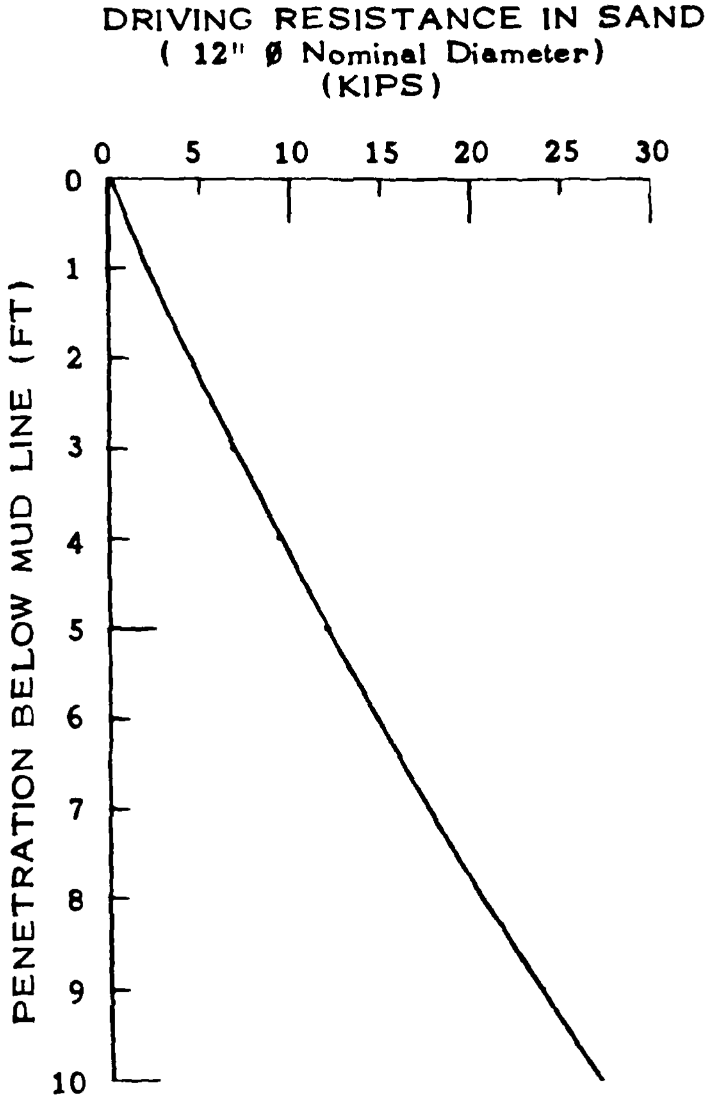


Figure 3-5 Fender Pile Driving Resistance in Sand

CHAPTER 4. LIMITING STRENGTH OF EXISTING POL LOADING PLATFORM

4.1 Introduction

The structural strength of the existing POL loading platform is evaluated in this section. In the process of structural analysis, the platform structure is approximated by a series of plane frames spaced evenly along the longitudinal axis of the platform. The plane frame structure consists of a horizontal concrete T-beam and four steel pipe columns. Two exterior columns are vertical and two interior columns are inclined at 2.5 to 1 slope in an opposite direction. The four columns are embedded into the T-beam stem. Section A-A in figure 2-5 illustrates the beam-to-column connection of the plane frame. The four columns are assumed pinned at the mud-line. A horizontal load is applied to the frame at the concrete deck level.

4.2 Pile Cap Pull-Out Mechanism

When the horizontal load P_1 is gradually applied to the plane frame, the structure will deflect accordingly in the direction of the applied load. An exaggerated deformed shape of the plane frame is shown in figure 4-1a. Due to the deflections of the two interior columns are in the opposite direction, the concrete T-beam will be forced into a double-curvature deformation. As shown in this figure, the critical condition is at point C (or C') where the concrete T-beam is under positive moment and the steel pipe column is under tension. The maximum pull-out resistance of the column is 36,000 lbs (see § 2.3.2). The corresponding strength of the horizontal load P_1 is found to be

$$P_1 = 27,000 \text{ lbs.}$$

The structural analysis of the plane frame failed by the pile cap pull-out mechanism is included in Appendix D.3.

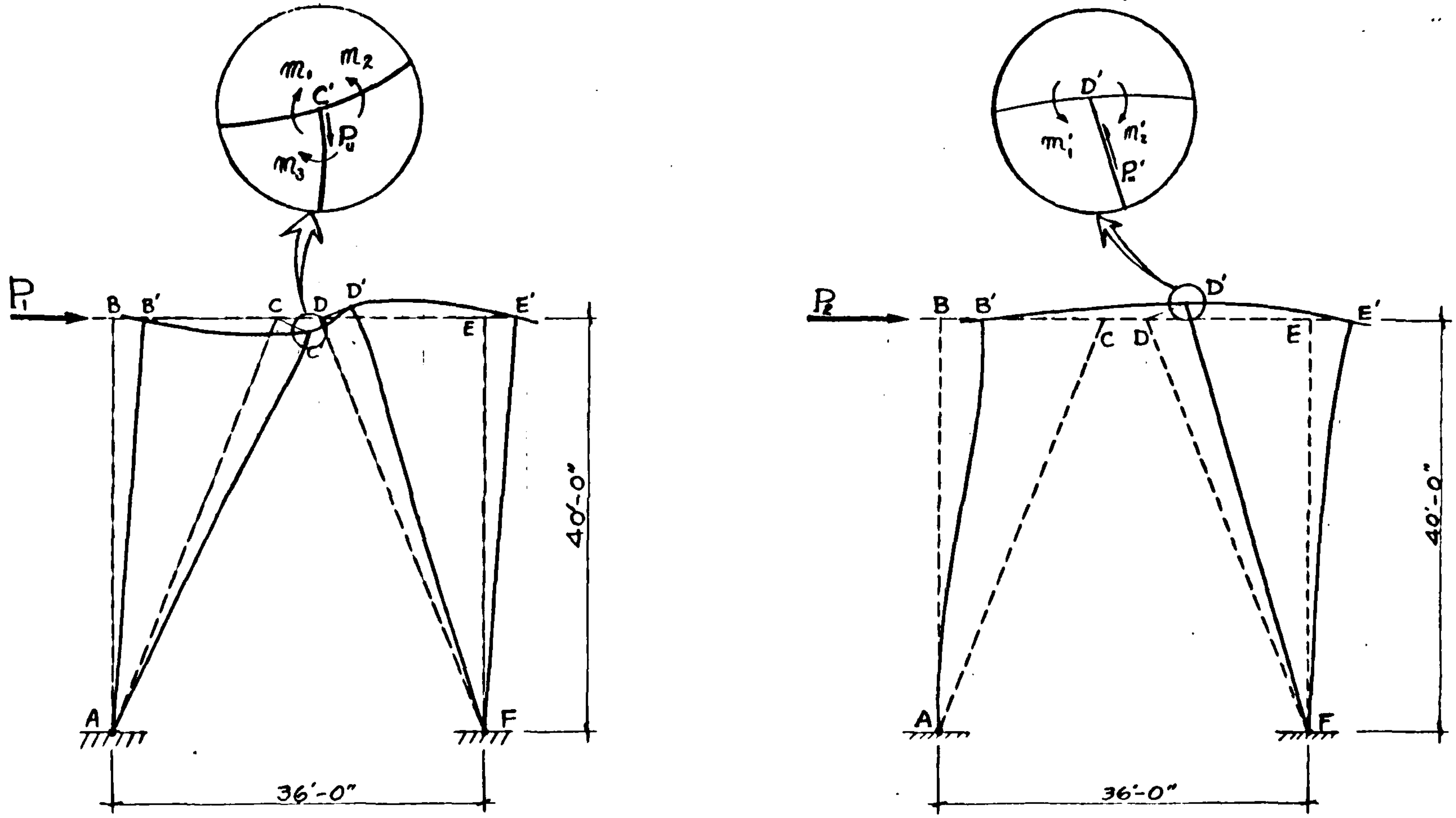
4.3 T-Beam Failure Mechanism

After the pile cap has been pulled out, the concrete T-beam will transform into a single curvature deformed shape, as shown in figure 4-1b. Under such circumstance, the concrete T-beam at point D location will be subjected to a negative moment. It is noted that the negative moment capacity of the T-beam is 424.6 ft-kips as discussed previously in §2.3.3.

The horizontal force P_2, limited by the tensile yielding of the T-beam strength is

$$P_2 = 39,400 \text{ lbs.}$$

Appendix D.3 also compiles the structural analysis of this case.



(a) Pile Cap Pull-Out Mechanism

(b) T-Beam Failure Mechanism

Figure 4-1 Possible Failure Mechanisms of POL Pier Structure

4.4 Berthing Velocity Limitation

The possible failure mechanism of the POL pier due to the impact loads of the berthing ships may be summarized as follows:

(a) Fender System

- Fender pile bending failure
- Lateral load resistance failure at the lower end of the piling

(b) Loading Platform Structure

- Pile Cap pull-out failure
- Tensile yielding of the concrete T-beam under negative moment

In order to prevent the above mentioned failure mechanisms from occurring to the POL pier structure, the limiting berthing velocity of ships is derived.

The kinetic energy for ship to jetty or quay service is calculated from the following relationship:

$$E = \frac{C_B W V^2}{2g}$$

where E = the kinetic energy absorbed by the pier system

C_B = berthing coefficient, 0.5 (see references 6 and 11)

V = berthing velocity

g = gravitational acceleration

W = Wa + Wb

Wa = added mass tonnage

Wb = actual displacement tonnage at the time of berthing operation

In most cases, the added mass tonnage can be approximated as 60% of the actual displacement tonnage, or

$$Wa = 0.6\ W_b$$

Thus, the energy equation may be rewritten in the form:

$$E = 0.4\ W_b\ V^2/g$$

or $$V = 360 \sqrt{E/W_b}$$

Where E = kinetic energy, ft-kips

W_b = actual tonnage, long-tons

V = berthing velocity, ft/min.

Figure 4-2 is the graphical illustration of the berthing velocity limitations for the POL pier at different water levels. The calculations in support of this figure are compiled in Appendices D.4 and D.7. It is noted that the solid lines shown in figure 4-2 represent the limiting strengths of the POL loading platform structure and fender system. A factor of safety should be imposed to the limitations for daily operation. For example, the factor of safety of 1.5 is imposed on the ultimate strength curve at EL. (-) 2'-0" water level, the allowable berthing velocity for a half-loaded T-5 tanker (26,000 L.T.) should be 10.4 ft/min, as illustrated in figure 4-2.

4.5 POL Pier Design Factors

Table 4-1 summarizes the design factors of the existing POL pier at Lajes Field, Azores and those required by the Naval Facilities Engineering Command (NAVFACENGCOM). It is noted that the existing POL pier does not conform to the NAVFAC design requirements.

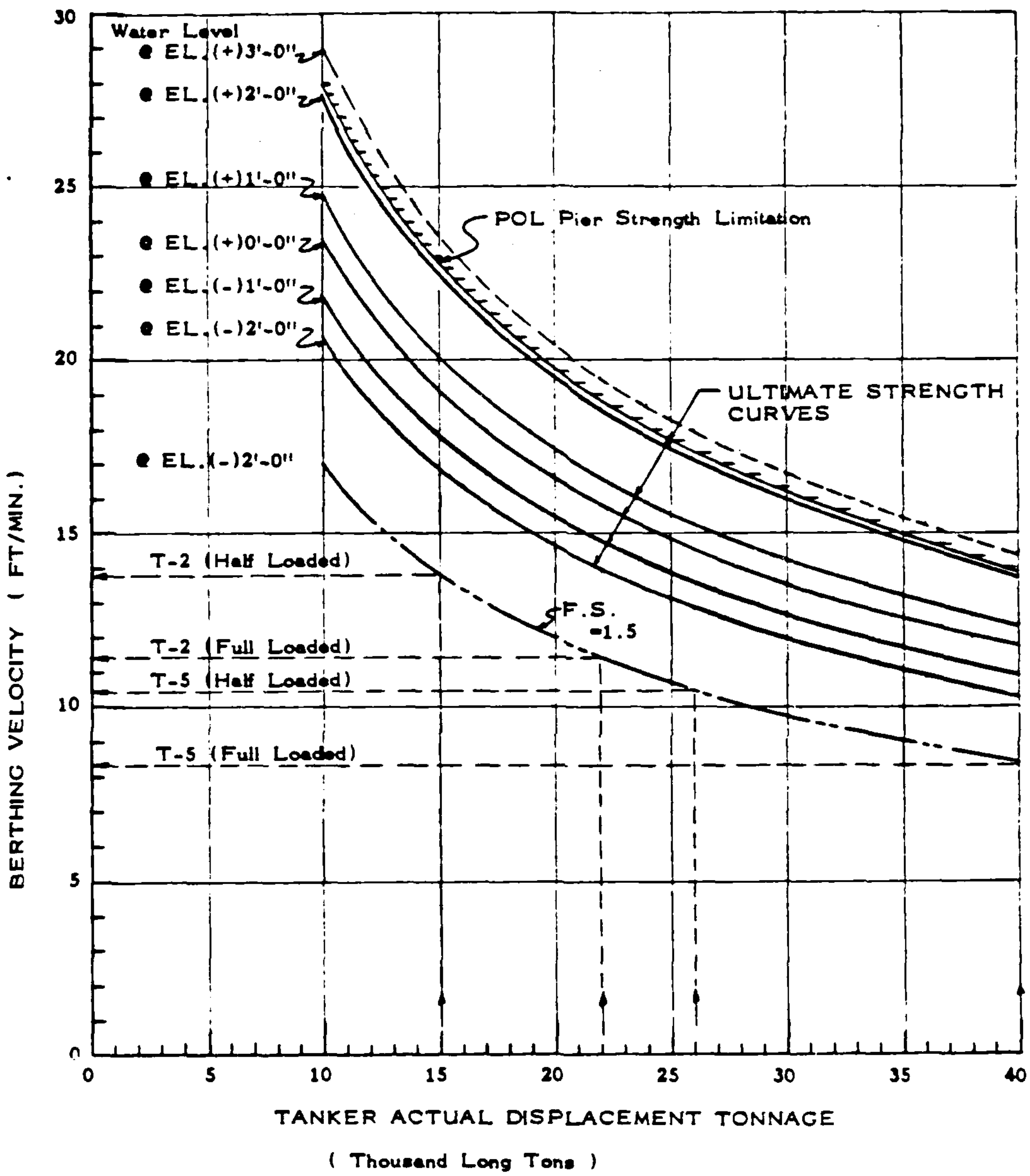


Figure 4-2 Berthing Velocity Limitation

Table 4-1 POL Pier Design Factors

DESIGN FACTOR	EXISTING POL PIER DESIGN CONDITION	NAVFAC POL PIER DESIGN REQUIREMENT
DIMENSIONS:		
Length (ft)	200	580* (T-2 Class) 735 (T-5 Class)
Width (ft)	40	40 (Minimum)
Water Depth (ft)	40	40 (Minimum)
VERTICAL DECK LOADS:		
Uniform Load (psf)	200	400
Truck Load	HS 20-44	HS 20-44
Impact	N.A.**	15% of truck load
Crane Load (tons)		
Total	N.A.	40
Deck Design Load	N.A.	75% wheel load
Piles & Pile Caps	N.A.	100% wheel load
Impact	N.A.	20% wheel load for deck 10% wheel load for pile cap and deck beam

* Length of vessel + 40 ft at each end
** Not Available

Table 4-1 (Continued)

DESIGN FACTOR	EXISTING POL PIER DESIGN CONDITION	NAVFAC POL PIER DESIGN REQUIREMENT
LATERAL LOADS: Forces from Moored Ship (lbs/ft)	N.A.	1,500 (Minimum) 2,700 (Acceptable) 4,000 (Maximum)
Berthing Velocity (ft/min)	15 (T-2 Class)	Exposed Pier: 42 (T-2 Class) 36 (T-5 Class) Sheltered Pier: 15 (T-2 Class) 12 (T-5 Class) Recommended Values for Lajes Field POL Pier*** 30 (T-2 Class) 24 (T-5 Class)
Wind Velocity (MPH)	65	65

*** CHESNAVFACENGCOM(FPO-1) recommendations based on variation of wind directions at Lajes Field POL Pier site.

Table 4-1 (Continued)

DESIGN FACTOR	EXISTING POL PIER DESIGN CONDITION	NAVFAC POL PIER DESIGN REQUIREMENT
LONGITUDINAL LOADS:		
Truck Load	N.A.	5% of total truck live-loads on any track without impact
Crane Load Braking	N.A.	15% of live-loads on any track without impact
Traction		25% weight on driving wheels on any track without impact
REFERENCES:	File No.7571-5799 (Appendix B.1 of this report)	NAVFAC DM-25 DM-26 (References 2 & 11 of this report)

CHAPTER 5. ALTERNATIVE APPROACH I --LOADING DOLPHIN SYSTEM

5.1 Introduction

The main concept of the loading dolphin system is to construct three lateral load-resistant dolphins to prevent the loading platform from direct contact with the berthing ships. Figure 5-1 depicts the proposed system. It is noted that the system shown in figure 5-1 is similar to the POL Pier Modification originated by the Army Corps of Engineers, New York District in June, 1966 (See Appendix B.2). The added dolphin (No. 3) is for the accommodation of the T-5 class tankers.

5.2 Loading Dolphins

Lateral load-resistant dolphins will be subjected to impact loads at water line level induced by the berthing ships. Hence, the functions of the dolphins will be to resist the horizontal impact force and the corresponding overturning moment. Depending on the methods of counter-balancing the applied loads, the following two types of dolphins may be constructed in this system.

5.2.1 Friction Resistance Type Dolphins

Figure A-3 of Appendix C, illustrates the basic components of the friction resistance type dolphins. Steel sheet pilings will first be driven to the rock bed to form a cofferdam and then filled with stones. A hung fender system above the water level will be attached to the dolphin.

The horizontal load applied to the dolphin will be resisted by the friction force between the sand surfaces at the cofferdam base level. The overturning moment will be counter-balanced by the gravity of the backfills. Appendix D.8 compiles the calculations of the approximate size of the dolphins for berthing T-5 class tankers.

5.2.2 Energy Absorption Type Dolphins

The energy absorption type dolphins are commonly constructed of steel H-shapes or pipe piles to form a space frame to resist the lateral load and the overturning moment. A typical profile of the dolphin is shown in figure A.4 of Appendix C. As shown in this figure, the vertical member will be driven through the sand layer to penetrate into the bed rock. The penetration into the bedrock will require drilling and grouting in the constuction process. Similiarly, an inclined member will also be installed to grade. These two members will then be connected at the top ends by means of a special linking mechanism so that the vertical member will be subjected to bending only. A completed dolphin will consist of at least a pair of the frames shown in figure A.4 of Appendix C and a hung fender system.

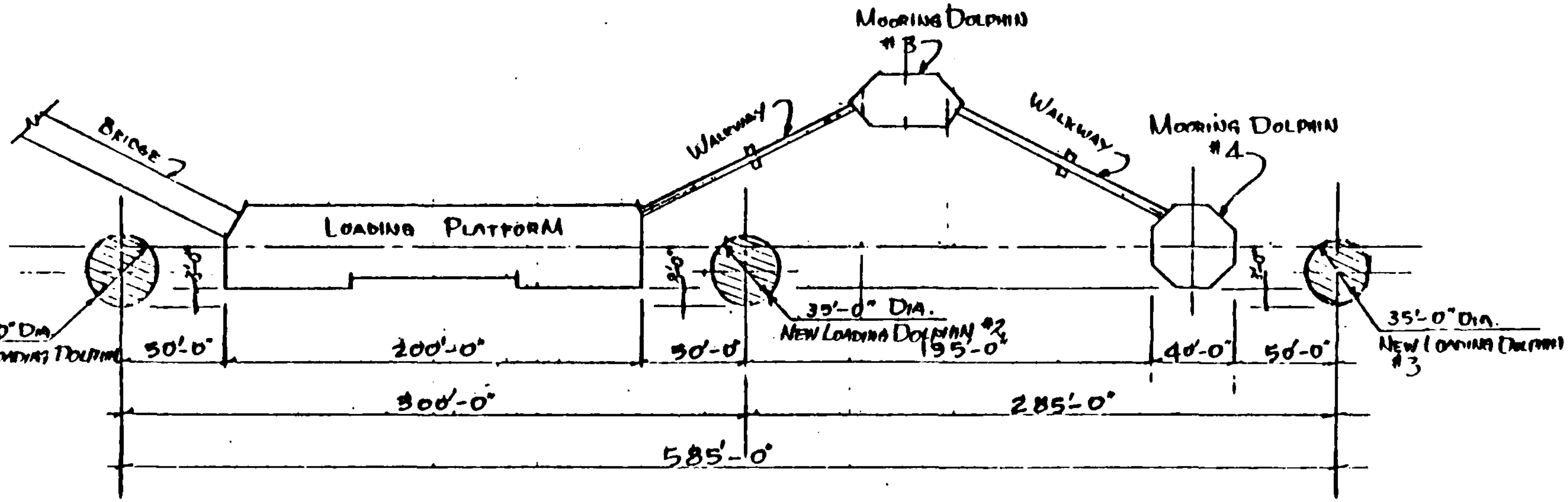


Figure 5-1 Loading Dolphin System

CHAPTER 6. ALTERNATIVE APPROACH II
--SINGLE BUOY MOORING SYSTEM

6.1 Introduction

The single buoy mooring (SBM) system was introduced into offshore industries about 20 years ago. The first SBM system was designed for 35,000 DWT tankers moored at approximately 100 feet water depth. As tanker sizes increased rapidly, existing berthing facilities proved inadequate. That was the big opportunity for SBM to develop. Today, SBM's are installed which can receive tankers up to 750,000 DWT.

The SBM system, as shown in figure 6-1, basically consists of a circular buoy with a diameter varying from 30 feet to 50 feet, anchored to the seabed by means of four, six or eight chain legs and fixed to the bottom either by conventional anchors, driven piles or drilled-in piles.

On top of the buoy, a turntable is mounted on a roller-bearing allowing a 360 degree rotation. This turntable is fitted with pipings, valves, mooring arm, floating hose connections, navigation aids and, in most cases, lifting equipment.

The center of the buoy body houses the central swivel essential for fluid transfer between fixed and rotating parts of the buoy.

Usually, the bottom connection to the pipe-line manifold is made by one or more hose strings. Floats are fitted to the underbuoy hose strings to obtain a smooth curve between pipe-line end manifold and the buoy.

The tanker is moored to the turntable mooring arms by thick nylon ropes. Oil transfer is by way of one or more floating hoses.

It has been proven that large tankers have an economic use superior to that of their smaller counterparts. However, proliferation of large carriers was rendering many of the world's traditional ports obsolete. It was also causing public concern because of hazards to port facilities and pollution risk. These unfavorable factors may be future critical problems for the POL pier at Lajes Field.

The tentative arrangement of the proposed SBM system at Lajes Field is shown in figure 6-2. The buoy will be anchored to the seabed at approximately 1 to 1 1/2 mile outside the new breakwater. A submarine pipe-line system will be installed to connect the buoy and the existing onshore piping system. The reason for the submarine pipeline to come onshore at the northern end of the breakwater is to avoid the possible conflict with future extension of the breakwater southward by other planners.

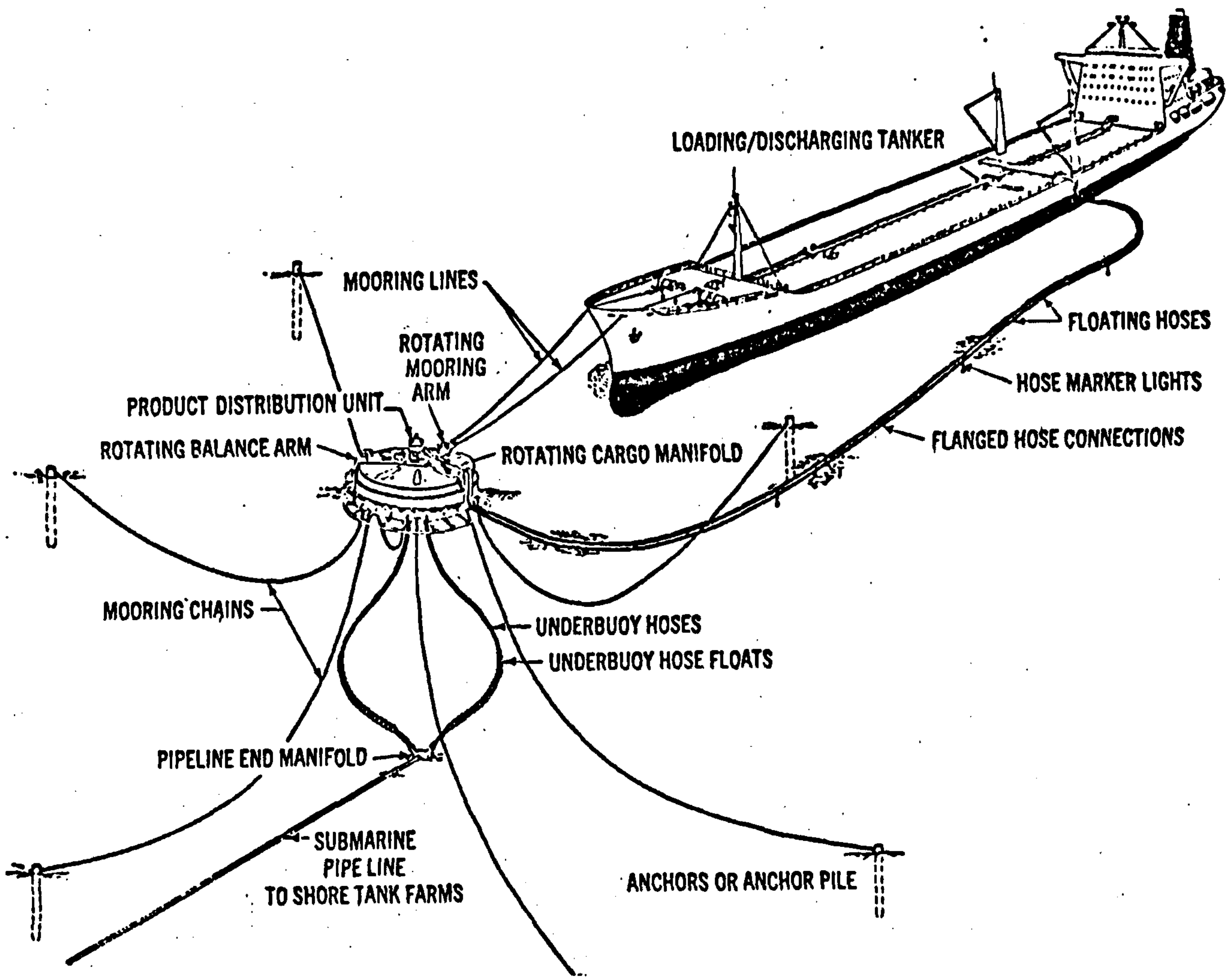


Figure 6-1 Single Buoy Mooring System

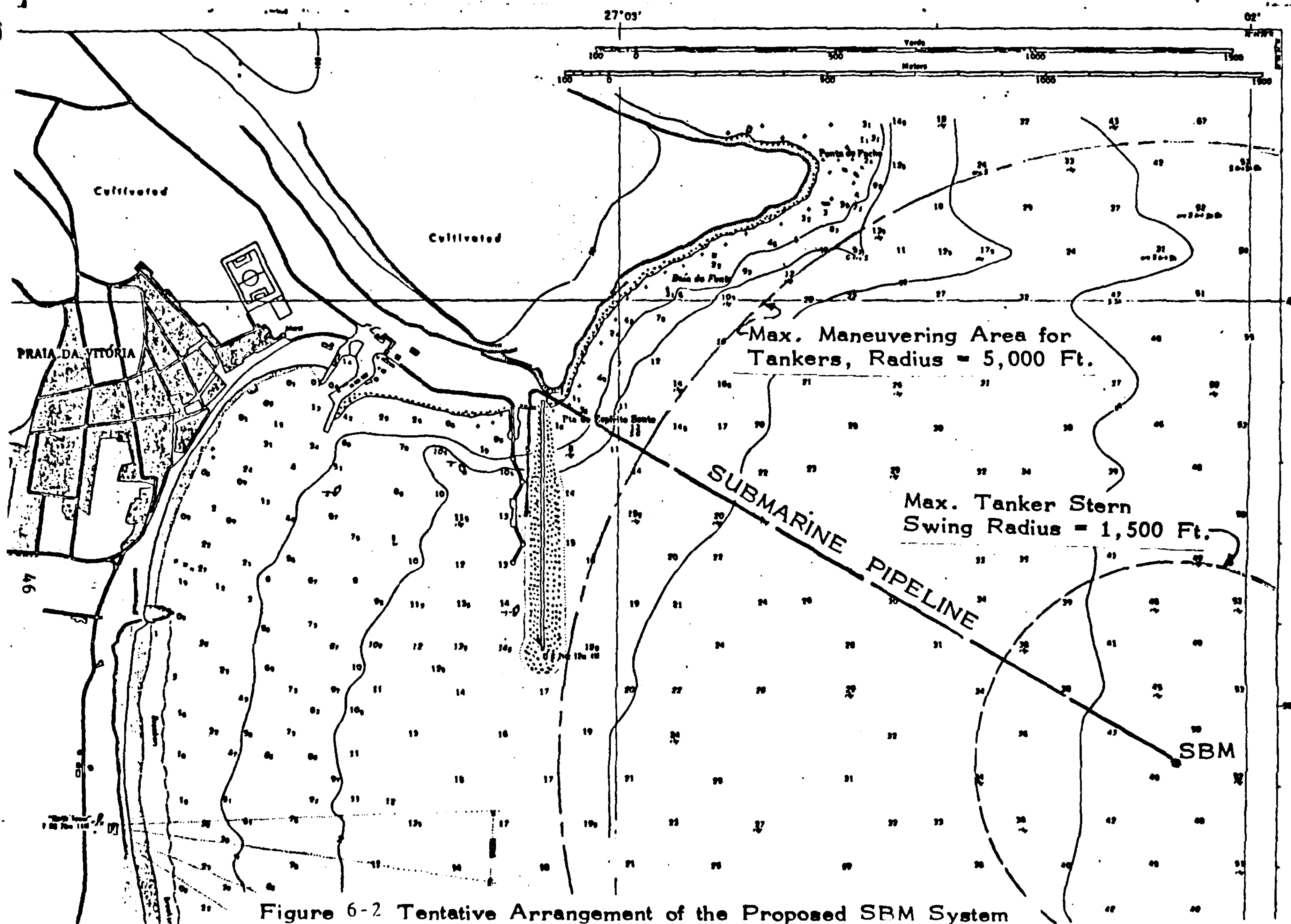


Figure 6-2 Tentative Arrangement of the Proposed SBM System

The proposed SBM system will possess the following distinct features:

- A Deep Water Terminal

As the tanker is moored to the SBM system, the ship will have a stern swing radius of 1,500 feet where the water is at least 100 feet deep. The system also provides a 5,000 foot radius of tanker maneuvering area at a minimum of 60 feet water depth.

- A Safe, Secure "Weathervane-Type" Mooring

A tanker moored to an SBM terminal is secured with bow lines only and is free to rotate around a 360-degree arc, like a weathervane, always heading into the wind, sea and current. The tanker and terminal are not subject to the environmental forces and contact imposed on a tanker rigidly secured to a pier.

- A Flexible Installation

The SBM system can be installed at almost any point off any coastline or in any navigable water. It can also be relocated should the need arise.

- Reduces or Eliminates Pilotage and Tug Costs

Tugs are not required for berthing vessels to an SBM system. By eliminating conventional harbor entry and mooring arrangements, tanker handling costs are reduced.

- Reduces or Eliminates Maintenance Costs on Existing POL Pier

The operation of the existing POL pier at Lajes Field will involve continuous efforts on the maintenance of the fender system, complicated and expensive modifications of the loading platform structure, and expensive dredging of large amounts of sediment inside the harbor. By installing the SBM system, the maintenance costs on the existing POL pier will be reduced considerably.

6.2 SBM System Components

6.2.1 Buoy

The buoy is the core of the SBM system which keeps the whole structure afloat. A field trip was made by LT. Wright (FPO-1) to investigate the condition and availability of the Government owned buoy for possible use in the SBM system at Lajes Field, Azores. The findings are:

- There are two buoys on the west coast which are part of the Consolidated Equipment Support Office (CESO) inventory.

- One of the buoys is made of steel by J. Ray McDermott. The buoy is in excellent condition and currently located at Port Hueneme, California. Figure 6-3 shows the buoy stationed on shore. The buoy weighs 150 tons. A complete inventory of chain and anchors is available.

- The other buoy is also made of A36 steel by IMODCO but weighs approximately 113 tons. Figure 6-4 shows the components of the buoy. The buoy is located in water at Coronado, California. The buoy is in good condition and can be upgraded with new teflon bearings in the swivel connection. CESO indicated that this buoy is the most eligible for turnover to USAF.

6.2.2 Anchor Chains

The SBM terminal adopts a multi-chain system to effect anchoring of buoy body. Normally, a four-point or a 6-point mooring has proved stable.

To a great extent, the number of chains determines a SBM. Thus as the amount of chain increases, buoy size will have to be increased in order to bear the extra weight. In order to minimize wear which is brought about by wave action, the chains are pretensioned with a load between 10 and 25 tons.

The principle of SBM's anchorage system is based on an equilibrium relationship between chain weight per unit length and excursion. More specifically, when a mooring force is applied, the buoy will move sideways, thus lifting a certain length of chain off the seabed. When weight of lifted chain is sufficient to balance the mooring force a new equilibrium is found, and no further excursion occurs until the load is changed. In order to prevent the chain from coming to a fully tensioned position, causing shock loads and possible breakage, the chain length should be sized so that it remains on the seafloor under the maximum excursion at the maximum design load.

Conventional anchors, gravity anchors, driven piles, and drilled-in piles all have been used as the chain anchor points.

6.2.3 Submarine Pipeline and Risers

These shall be designed, constructed and installed to provide the greatest possible fuel throughput.

6.2.4 Pipe-Line-End Manifold (PLEM)

The PLEM is connected to the end of the submarine pipeline and secured to the seafloor by either driven piles or drilled-in piles. The PLEM and buoy are then connected by flexible underbuoy hoses.

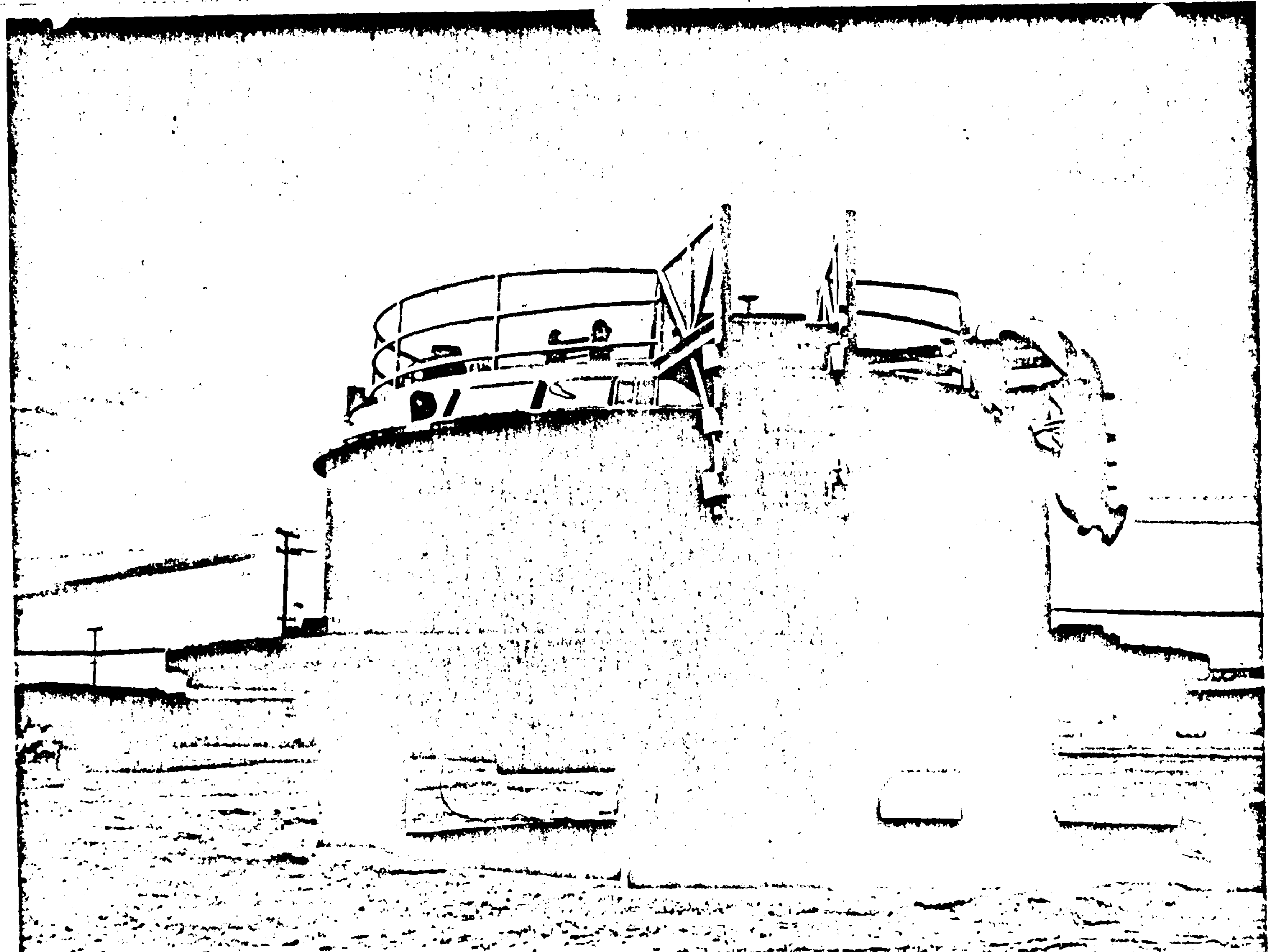


Figure 6-3 McDermott Buoy at Port Hueneme, CA

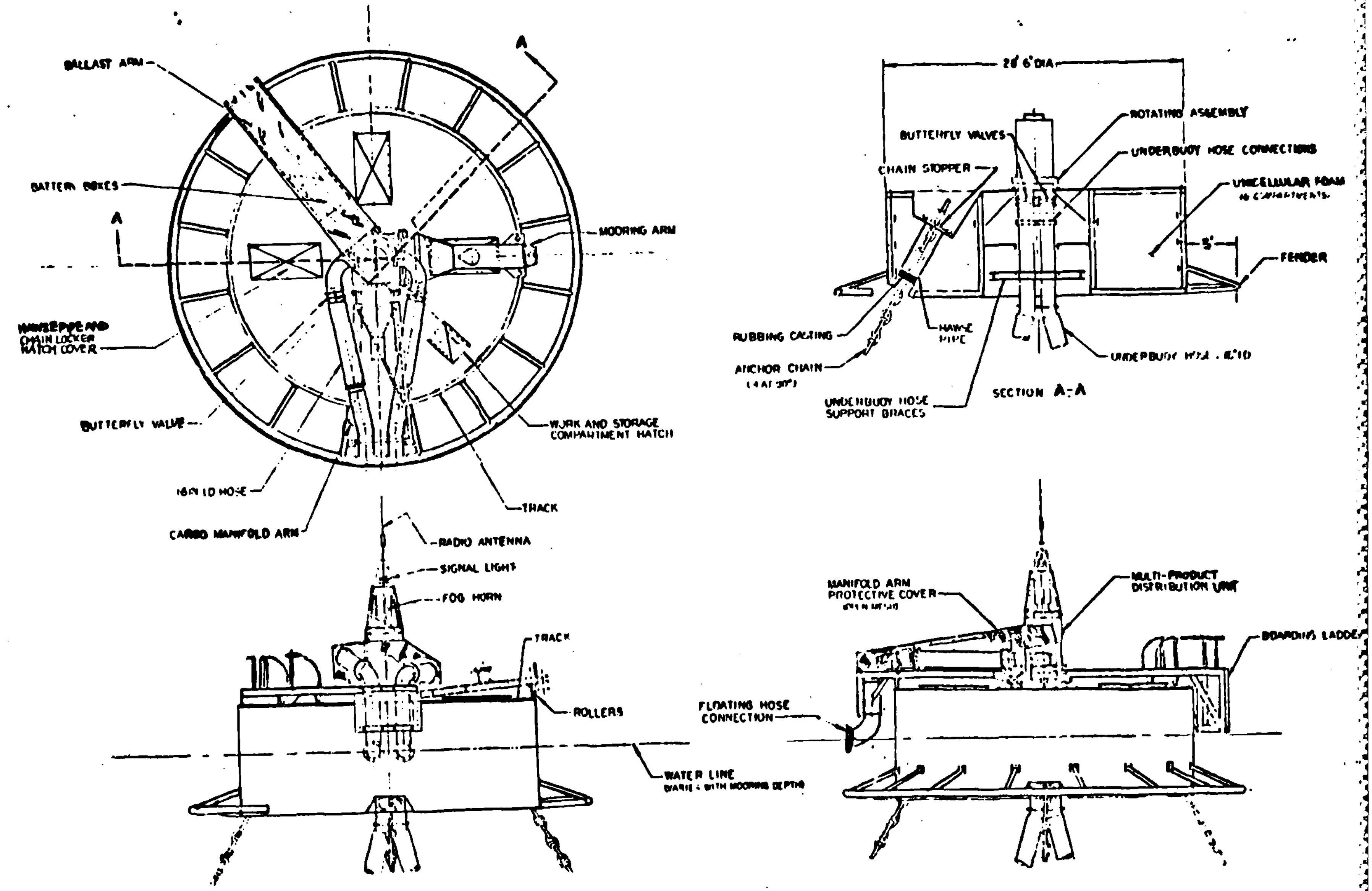


Figure 6-4 IMOCO Buoy at Coronado, CA

CHAPTER 7. SUMMARY AND RECOMMENDATIONS

7.1 Summary

The engineering evaluations of the POL pier fender system and loading platform structure at Lajes Field, Terceira, Azores are summarized as follows:

- A new fender system is needed to replace the original fender system which has deteriorated due to operational damage and biological attack. A fixed wooden pile fender system with cylindrical rubber fenders will improve the energy and force transferring function of the POL pier system. However, a new fender system will contribute little to the reinforcement of the pier system in accomodating up to 40,000 DWT tanker operation.

- Spalling concrete around the pile caps in the south bent of the loading platform were due to the tensile force in the piling in excess of the bond force between the pile surface and the concrete. The pile cap pull-out mechanism represents the first stage failure of the platform structure subjected to a lateral load applied at the concrete deck level. The result of this failure will not cause the structure to collapse. However, the additional load beyond the first stage failure load will be transferred to the concrete deck beam and eventually will cause the beam to fail.

- Cracks in the south bent concrete deck along the longitudinal axis of the loading platform were caused by the negative moment acting on the effective T-beam section of the concrete deck beam in the mid-span location. A continuous yeilding of the top reinforcing steel of the concrete deck is the second stage failure of the platform structure. The yield strength of the concrete T-beam under negative moment is the ultimate strength of the platform structure.

- Epoxy or cement grouting of the concrete spalling and cracks will not increase or reinforce the strength of the existing platform structure. However, the grouting will prevent the deck reinforcing bars and pile cap steel surfaces from environmental corrosion.

- The existing platform structure does not possess sufficient lateral load resistance capacities for tankers in the 40,000 DWT size to operate under normal berthing conditions. To increase the structural strength of the loading platform, the attention shall be in the preventation of the pile cap pull-out mechanism and the reinforcement of the effective T-beam section of the concrete deck.

- One of the alternatives to a structural modification of the pier is the construction of three dolphins to divert the ship berthing energy and force from the loading platform. This approach appears to require less initial capital investment. However, the maintenance of a ship approaching channel and the turning radius inside the harbor will be continuous expensive operation.

- The Single Buoy Mooring (SBM) system approach is another alternative. The SBM system will require the installation of an off-loading buoy and submarine pipe line outside the breakwater. The systems requires a relatively high initial capital investment. However, the convenience of tanker off-loading, accommodation of tankers up to 200,000 DWT sizes, and near trouble free maintenance of the system may have a long range benefit as compared with the others.

7.2 Recommendations

The recommendations for the future work on the POL pier system are:

(a) Fender System

- Design a new fender system to accomodate T-2 class tankers or C-2 class cargo vessels.

(b) Platform Structure

- Repair spalling concrete and tension cracks with epoxy or cement grouting.

- Conduct a feasibility study to consider the operational and economical aspects of the POL system for tankers up to 40,000 DWT size and possibly up to 200,000 DWT sizes. The study shall include, but not be limited to, the following approaches:

  --reinforcing the existing platform structure
  --constructing a loading dolphin system
  --installing a SBM system

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APPENDIX A

Project Correspondence

This section contains three messages:

- Request of Assistance on the Structural Evaluation of the POL Pier (HQ MAC Scott AFB IL/DEM 132030Z Jan 78)
- Tasking Statement from MAC HQ
- Funding Information (1605 ABW Lajes FLD AZ 251430Z Jan 78)

ZNR UUUUU
R 132030Z JAN 78
FM HQ MAC SCOTT AFB IL/DEM
TO RUEBJHA/CHESNAVFACENGCOM WASH DC
INFO RULSSAA/COMDR NAVFACENGCOM ALEXANDRIA
JDLAAA/1605ABW LAJES FLD AZORES/CC/DE.
RUEOLFA/CINCLANT NORVA/J413
BT
UNCLAS

ACTION CHESA

SUBJ: STRUCTURAL INVESTIGATION OF POL PIER, LAJES FLD
1. REF 12 JAN 78 TELECON BETWEEN YOUR CMDR ERCHUL AND OUR MR. WEISSERT IN WHICH WE DESCRIBED VARIOUS STRUCTURAL PROBLEMS CURRENTLY BEING EXPERIENCED AT LAJES FLD. SPECIFICALLY, THE PROBLEM INVOLVES STRUCTURAL DEFICIENCIES AND DAMAGE TO THE POL PIER LOCATED IN PRAIA HARBOR. THE CONCRETE DECK AND STRUCTURAL CONCRETE ARE SPALLING. THE FENDER SYSTEM INCLUDES VERTICAL WOOD PILINGS ADJACENT TO THE CONCRETE PIER WHICH ARE CONTINUALLY SUBJECTED TO DAMAGE.
2. WE UNDERSTAND YOU HAVE PERSONNEL WITH MARINE STRUCTURAL ENGINEERING EXPERTISE AND ARE WILLING TO PROVIDE US WITH TECHNICAL ASSISTANCE ON AN IMMEDIATE BASIS. ACCORDINGLY, REQUEST THE FOLLOWING:

14 JAN '78 15:22

PAGE 02 RUCIMAA5651 UNCLAS
A. AN ON-SITE VISIT BY A MARINE STRUCTURAL ENGINEER.
B. A TECHNICAL EVALUATION AND ANALYSIS OF THE EXISTING SITUATION.
C. A REPORT OF RECOMMENDATIONS SHOWING WHAT STRUCTURAL MODIFICATIONS MUST BE MADE TO THE PIER IN ORDER TO ACCOMMODATE POL AND CARGO SHIPS OF THE 40,000 TON SIZE.
3. REQUEST THE ABOVE TECHNICAL ASSISTANCE BE PROVIDED AT THE EARLIEST POSSIBLE TIME. ASSUMING YOU CAN FULFILL THE REQUESTS CONTAINED IN PARA 2 ABOVE, WE WILL MAKE ARRANGEMENTS TO HAVE OUR MR. WEISSERT (AUTO 638-3067/2740) MEET WITH YOUR REPRESENTATIVE ON SITE. YOU SHOULD MAKE ALL TO AND FROM TRANSPORTATION ARRANGEMENTS FOR YOUR REPRESENTATIVE. AREA CLEARANCE SHOULD BE OBTAINED IAW USAF FOREIGN CLEARANCE GUIDE. ON BASE VOQ ACCOMMODATIONS ARE AVAILABLE.
4. IT IS OUR UNDERSTANDING THAT CHARGES FOR THE ABOVE REQUESTED SERVICE FOR A PERIOD OF UP TO TWO WEEKS WILL BE ON A REIMBURSABLE BASIS AND WILL NOT EXCEED $5,000. ACCORDINGLY, THE 1605 ABW AT LAJES WILL ISSUE A MILITARY INTERDEPARTMENTAL PURCHASE REQUEST (MIPR) IN THIS AMOUNT WHEN PLANS ARE FIRM.
FOR 1605: WE WILL FURNISH THESE FUNDS TO YOUR HQS.
BT 132030Z JAN 78 DO
#5651

05 FPO-1 TA 1E 1C COPY

AUTO 690-4546 LANTDIV - FOR 05A22B, PAUL DAVIA

DEPARTMENT OF THE AIR FORCE
HEADQUARTERS 1605th AIR BASE WING (MAC)
APO NEW YORK 09406

REPLY TO ATTN OF: DE

SUBJECT: Praia Port POL Pier

TO: MAC/DE

1. Construction of the Praia port POL pier was completed in early 1963. The pier was constructed to accommodate a T-2 tanker (length - 524 ft., displacement 21,800 tons) moored with a 65 mph wind acting on the tanker and berthing with a normal component of approach velocity of 15 fpm. Due to a general increase in the size of current ships, ships as large as A0143 class tankers (length - 656 ft, displacement 38,000 tons) are now using the port. (Note: Other ship specifications also relate to the effect on the pier by the ship. Length and displacement tonnage have been shown only for comparison.) The pier has deteriorated since its original construction thus reducing the structural capacity of the pier by an undetermined amount. The harbor area is relatively exposed, also the actual impact berthing velocity is suspected to be considerably greater than 15 fpm due to exposure conditions and operational limitations. Thus, due to the increase in ship size, deterioration of the pier, exposed location, and possible higher berthing velocities, the pier's capability to meet operational requirements is subject to question.

2. Deterioration of the pier includes general biological attack and operational damage to the protective fender system and operational damage, corrosion and patterned deterioration of the pier structure itself.

a. Deterioration of the fender system is due to biological attack of the timber piles and berthing and mooring operations causing impact loading, cyclic loading, and friction on the timber members. Within a short time after original construction, such damages necessitated repairs which resulted in reconfiguration of the entire timber portion of the fender system. The fender system originally consisted of vertical timber piles, rigidly connected by horizontal chocks and whales with rubber blocks between the timber members and the concrete deck of the pier. To facilitate repairs in limited periods of time and during adverse weather conditions, the horizontal chocks and whales were supplemented and eventually replaced by continuous timber poles on both sides of the replacement vertical piles with cable connecting the timber members. This reconfiguration has not significantly reduced the energy absorption capacity of the fender system since the timber members absorb an insignificant amount of energy relative to the energy absorption of the rubber blocks. However, the fender system in its present configuration is more susceptable to damage from oscillatory motions of a moored ship. Such damage consists of the vertical timber piles working out of the harbor bottom, sawing of the timber members by the cables, and splintering of the timber members near their ends due to compression and torsion. This damage increases maintenance and repair.

b. Deterioration of the pier structure consists of: limited corrosion of the steel piles which support the pier, structural cracks in the concrete deck and concrete beams which have resulted from at least one major accidental ship impact, corrosion of reinforcing in the concrete deck and concrete beams due to the cracks and marine atmosphere, and patterned spawling of the concrete around the ship-side center pile of each bent in the south section of the loading platform. Each component of this deterioration may have reduced the capability of the pier to withstand loads imposed by berthing or moored ships. Additionally, the patterned spawling indicates a possible over-stressing of the pier, which in view of the increased size of ships utilizing the pier, is highly probable.

3. Repair of the fender system has become a continuous operation. Fifteen to forty timber piles are used each year depending on weather conditions and harbor operations. Forty new timber piles arrived at Praia Harbor on 28 Nov 77. These are being used to replace significantly rotted and damaged timber members. Approximately 25 of the piles are needed now and the remainder will be used as required. Each pile costs $650.00. An average of 1250 MH per year has been expended each year for the past three years. This equates to the equivalent of a complete fender system (timber portion) replacement on an average of every three years.

4. Repair and maintenance to the pier structure now underway consists of AZ 77-0080, Corrosion Control POL Pier; AZ 77-0079, Replace Floodlights and Handrails POL Pier; and AZ 76-0031, Install Anode Beds. Projects AZ 76-0031 and AZ 77-0079 will prevent corrosion of the supporting steel piles which will prevent further structural deterioration of the steel piles.

5. My engineering staff has been tasked to study the possibility of installing a new fender system which will reduce maintenance and increase the energy absorption capacity of the fender system to a level commensurate with present operations. However, an extensive structural analysis of the pier is required to determine the capacity of the pier structure. I question whether the pier structure has the capability to meet present operational requirements even with a new, improved fender system. This evaluation is required immediately due to extremely high maintenance costs, the impact on Civil Engineering, and the possibility of major damage occurring to the pier structure or to ships. The pier is critical to the U. S. Forces, Azores operations.

6. Additionally, as a matter of information, the Portuguese Government is planning extensive construction in the Praia Harbor area, including the possibility during a second phase of construction of extending the pier. Plans indicate initial construction will begin in late 1978, although we have nothing that would suggest that this date would be met.

7. For the present, we are requesting your support in obtaining engineering assistance to evaluate the structural capacity of the Praia Port POL Pier, inclusive of its fender system. The evaluation will require expertise in marine structural engineering.

8. The attached photographs, narrative of the photographs, and drawing of the pier further describe the existing conditions.

RICHARD T. DRURY, Brig Gen, USAF
Commander

3 Atch
1. Narrative of Photographs
2. Photographs (24 ea)
3. Drawing 88-01-03, Sheet 19 of 38 w/comments

Cy to: MTMC
TTGE
Rotterdam, Netherlands
APO New York 09159

NARRATIVE OF PHOTOGRAPHS

DESCRIPTION OF PHOTOGRAPHS BY PHOTOGRAPH NUMBER

1. Cracks in concrete deck at south end of loading platform. Cracks appeared after ship impact in 1972, and were surface epoxy grouted afterwards. Cracks extend through deck (See photos 4,6,8,9 & 10).

2. Cracks in south end face of loading platform. (Point of ship impact described in photo #1.) Note: Rust and salts leaching from cracks.

3. Spalled concrete around ship-side center pile in bent #1.

4. Cracking in concrete deck, between bents 1 & 2 and in beam of bent #3.

5. Spalled concrete around ship-side center pile in bent #2.

6. Spalled concrete around ship-side center pile in bent #3, and cracking in deck between bents 2 & 3 and in beam of bent #3. Note: Extensive rust and salts leaching from cracks, and difference in condition of concrete at piles.

7. Spalled concrete around ship-side center pile in bent #3. Limited corrosion of steel pile.

8. Spalled concrete around ship-side center pile and crack in beam of bent #4. Note: Difference in condition of concrete at piles and rust and salts leaching from crack.

9. Spalled concrete around ship-side center pile of bent #4 and cracking in deck between bents 4 & 5 and in beam of bent #4. Note: Rust and salt leaching from cracks.

10. Cracking in bottom side of concrete deck between bents 4 & 5. Note: Rust and salt leaching from cracks.

11. Spalled concrete around ship-side center pile of bent #5. Note: Sections of concrete ready to fall.

12. Spalled concrete around ship-side center pile of bent #5. Note: Section of concrete ready to fall.

13. Typical bent in center section of loading platform is in good condition.

14. Bents in north section of loading platform are in good condition.

15. Limited corrosion of typical steel pile.

16. Deteriorated protective timber pile cluster at north end of loading platform. Note: Deterioration at water line. (Also note: Arrangement of steel piles as reference for photos 1 through 14.)

17. Same as photo #16 - closer view.

18. Deteriorated protective timber pile cluster at south end of loading platform. Note: Deterioration at water line. (Also note: Cracking and spalling of concrete described in photos 2 & 3 and arrangement of steel piles as reference for photos 1 thorugh 14.)

19. Same as photo #18 - closer view.

20. Present configuration of fender system along loading platform.

21. Same as photo #20 - side view. Note: Cable used for connecting timber members.

22. Deteriorated and disconnected fender tie back chains used in original configuration of fender system.

23. Sawing of horizontal timber member by cable movement and splintered ends of vertical piles.

24. Fenders on south mooring dolphin. Note: Closest face of mooring dolphin is still protected with original construction fender system (this is tho only remaining portion of the original fender system).

NOTES ON PHOTOGRAPHS:

1. Spalling of concrete around piles exists at every ship-side center pile of each bent in the south section of the loading platform. (These piles act in tension during applied ship forces.)

2. Spalling of the concrete is non-existant in the center section, which does not receive forces from ships, and in the north section.

3. The large section of concrete dislodged at the ship-side center pile of bent #5 (See photos 11 & 12) indicates spalling is not occuring due to weathering or poor quality concrete but rather due to excessive structural movements. Also, spalling has not occurred anywhere else on the pier.

4. The cracking, running generally parallel to the longitudinal axis of the loading platform which resulted from the ship impact in 1972, coincides with the center piles which have not experienced spalling of the surrounding concrete (See photos 4,6,8,9 & 10). This indicates the ship impact of 1972, negligibly affected the spalling of the concrete around the ship-side center piles of bents 1 through 5.

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R 251430Z JAN 78 CHESDIV (A)
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TO RUEBJHA/CHESNAVFACENGCOM WASH DC
INFO RUCIMAA/HQ MAC SCOTT AFB IL/ACBC
RULSSAA/COMDR NAVFACENGCOM ALEXANDRIA VA
RUCBSAA/CINCLANT NORVA/J413
BT
UNCLAS
SUBJ: FUND CITATION - TDY
1. AUTHORIZATION IN THE AMOUNT OF $5,000 IS HEREBY GRANTED CHESNAVFACENGCOM WASH DC TO COVER COST OF TDY'S NECESSARY TO INVESTIGATE STRUCTURAL DEFICIENCIES AND DAMAGE TO POL PIER LOCATED IN PRAIA HARBOR AT LAJES FLD AZ. THE FOLLOWING FUND CITE IS APPLICABLE, 5783420 308 6505 244420 04 40710 40810 40910 S666200. CIC: 44 865 0444 666200.
2. THIS IS A COORDINATED 1605 ABW/ACR/ACFT/DE MSG. 251430Z JAN 78/KP
BT
#6581

ACTION

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09B		
09P		

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0143 REF

LEGEND:
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✓ — FURNISHED COPY

APPENDIX B

POL Pier Structural Drawings

B.1 Lajes Field Waterfront Improvement Drawings

The as-built drawings of the Lajes Field Waterfront Improvement, Lajes, Azores supplied by the Civil Engineer Squadron (CES) at the Lajes Field are tabulated as follows:

U.S.Army Engineer District, Eastern Ocean
Corps of Engineers
New York 13, N.Y.

Record Work As Built May 4, 1964

File No.	Sheet No.	Title
7571-5781	1	Location Plan & Drawing Index
-5782	2	General Arrangement
-5783	3	Borings & Seismic Exploration
-5784	4	Boring Logs
-5785	5	Meteorological Data
-5786	6	Tide, Sea and Swell Conditions
-5787	7	Observed Tide and Sea Conditions
-5788	8	Current Survey
-5789	9	Wave Record

File No.	Sheet No.	Title
7571-5790	10	Refraction Diagrams
-5791	11	Refraction and Diffraction Diagrams
-5799	19	POL Pier-General Arrangement
-5800	20	POL Pier-Bridges & Walkways
-5801	21	POL Pier-Pump House & Foundation
-5802	22	POL Pier-Loading Platform
-5803	23	POL Pier-Concrete Details
-5804	24	POL Pier-Mooring Dolphins Nos. 1&2
-5805	25	POL Pier-Mooring Dolphins Nos. 3&4 and Walkway Supports
-5806	26	POL Pier-Miscellaneous Details

B.2 POL Pier Modification Drawing

CES of the Lajes Field supplied the following drawing relative to the POL Pier modification:

U.S.Army Engineer District, New York
Corps of Engineers
New York 3, N.Y.

30 June 1966

File No.	Sheet No.	Title
--	--	Proposed Modification to POL Pier

APPENDIX C

Field Investigation Briefing

The notes documented herein are those presented to the officers at the command level in the Lajes Field on 3 February 1978.

POL Pier Field Investigation Briefing

Place: Lajes Field

Time: 0900 (Local Time)

Date: 3 Feb 1978

Attendant:

Base Vice Commander	Col. Bobby Massingill (AF)
Head Engineering & Design	Lt. Col. Larry England
Deputy	Lt. Col. Al Meyers
	Lt. Scott Fehseke
TTU Commander	Lt. Col. John Telfer (Army)
Deputy	Lt. Col. Walls
J-4	Lt. Col. Bentley
	LCDR. Allen Hill (Navy)
LG	Lt. Col. Rodenheiser
MAC HQ	Les Weissert
FPO-1	LT. James Wright
	C. Chern

LAJES FIELD

POL PIER INVESTIGATION

Ocean Engineering & Construction Project Office
Chesapeake Division
Naval Facilities Engineering Command
Washington, D.C.   20374

2 February 1978

SUMMARY

31 JAN 1978: IAW HQ MAC SCOTT AFB 132030Z JAN 78, Chesapeake Division personnel (Dr. C. Chern, LT J.C. Wright, CEC, USN) were briefed by CES and completed a surface inspection of the damaged POL pier. Completed photo documentation.

1 FEB 1978: Diving inspection of pilings completed on total length of pier system. Photo documentation completed on underside structural damages.

2 FEB 1978: Results of investigation analysed.

DIVE RESULTS

Time: 1030 Date: 1 FEB 1978
Visibility: 20 feet Average Bottom Depth: 45 feet
Air Temp: 65° F Water Temp: 55° F
Bottom Condition: Fine Sand; very little silt

Remarks: Very minor corrosion and marine growth on steel piles; overall condition of steel piles are excellent; bottom condition exhibits spread of debris, e.g. wire rope, broken timber piles near pier; old fender system is horizontally lodged among new timber piles.

PROBLEM STATEMENT

- Concrete pile cap on many interior steel piles has broken away due to horizontal loads applied in excess of design loads.

- Wood pile fendering system is easily damaged and components are being replaced at highly uneconomical frequencies.

RECOMMENDATIONS

NOTE: A complete report including detailed options and costs will be forwarded within 60 days. The following analysis and recommendations are provided for the interim.

FENDER SYSTEM

The present fendering system when repaired, is adequate for small boats such as an LCM at most wave conditions or larger ships only during very calm wind and wave conditions. The pier itself has not been designed for high impact loads and an improved fender system will not alter pier deficiencies.

IMMEDIATE Repair Recommendations:

- Utilize improved pile driving methods to insure that the base of the pile is stationary.
- Cut pile points for less driving resistance.
- Reduce vertical friction force by any of the following:

  1) Remove outer wale and grease vertical members.

  2) Remove outer wale and attach galvanized sheet metal to vertical members.

  3) As shown in attachment (1), utilize cylindrical rubber fender on timber facia. NOTE: Size of facia timber correlated to pile spacing.

  4) As per diving inspection, removal of broken piles and submerged horizontal fender is highly recommended.

Conclusion:

Fender Systems protect both ship and pier from minor impact damage and heaving abrasion damage. Therefore, since a fender is not utilized to resist high impact loads and must accept vertical friction loads, the removal of the outer wale is necessary. A fender system will be recommended only after the pier has been protected from high impact loadings.

PIER MODIFICATIONS

Attachment (2) - Docking for T-2 and T-5 Class Tankers

- Existing pier utilized for small craft only.
- Construction of two or three dolphins for use when docking T-2 class or T-5 class tankers.
- Dolphins provide protection to the pier from any direct horizontal loads. Typical dolphin systems are shown in attachments (3) and (4).
- Construction of dolphins would make the pier inaccessible for POL or cargo transfer for an estimated 30 to 60 days. Therefore, a temporary submarine pipeline can be constructed as shown in Attachment (5) for POL transfer.
- Some dredging may be required.

Attachment (6) - POL Transfer Utilizing SBM System

- Single Buoy Mooring (SBM) system is alternative to dolphin construction.
- POL transfer from tankers up to 200,000 DWT.
- Cost of new SBM system is estimated at $10 million.
- Existing POL pier utilized for small draft only.

NOTE: A third option consists of strengthening the existing POL pier to resist heavy impact loads. Costs would include structural modifications, heavy fendering system and dredging. This is not a recommended approach at this time.

ATTACHMENT (1)

2-2-78
FPO-1EA21

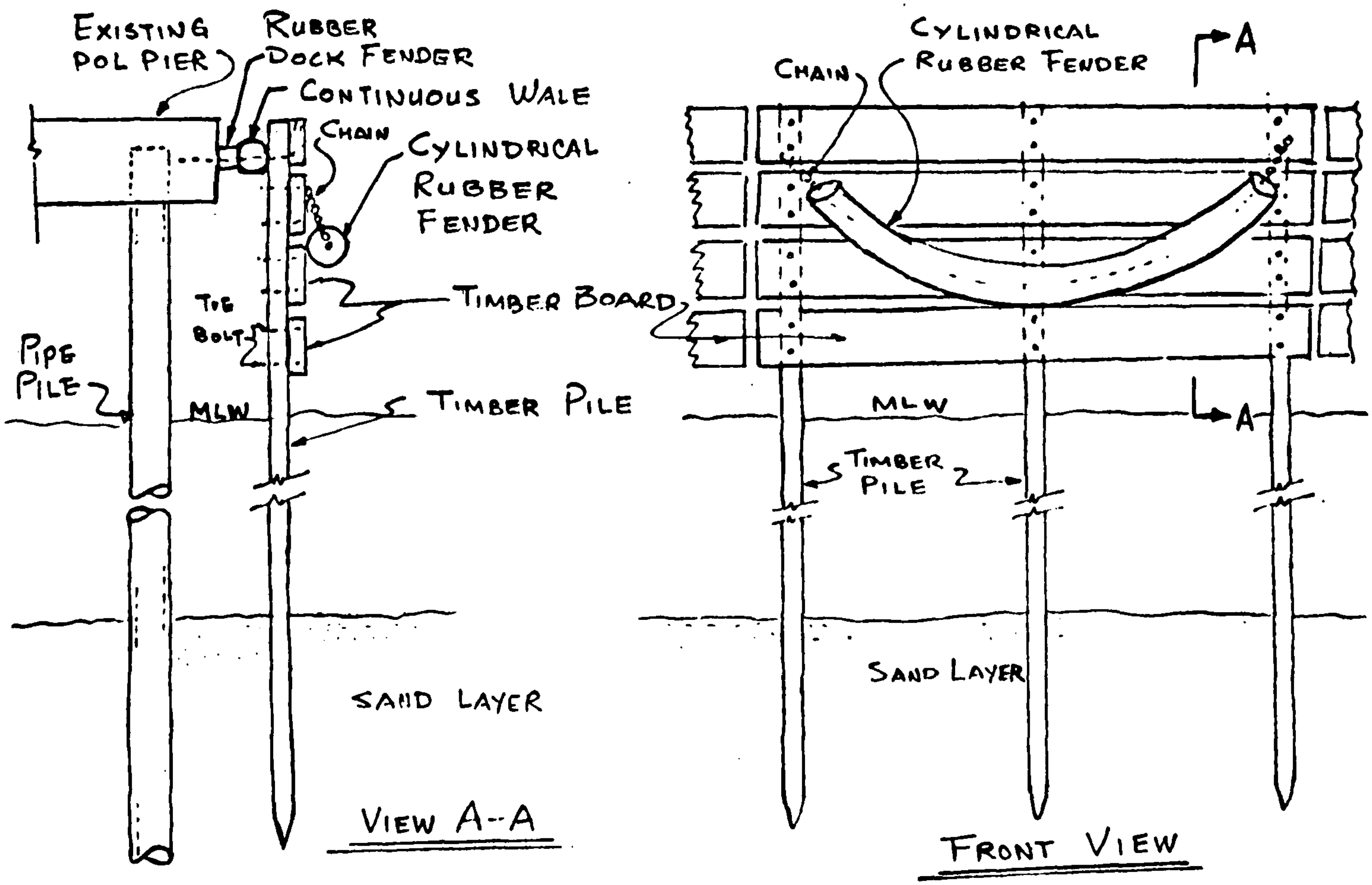
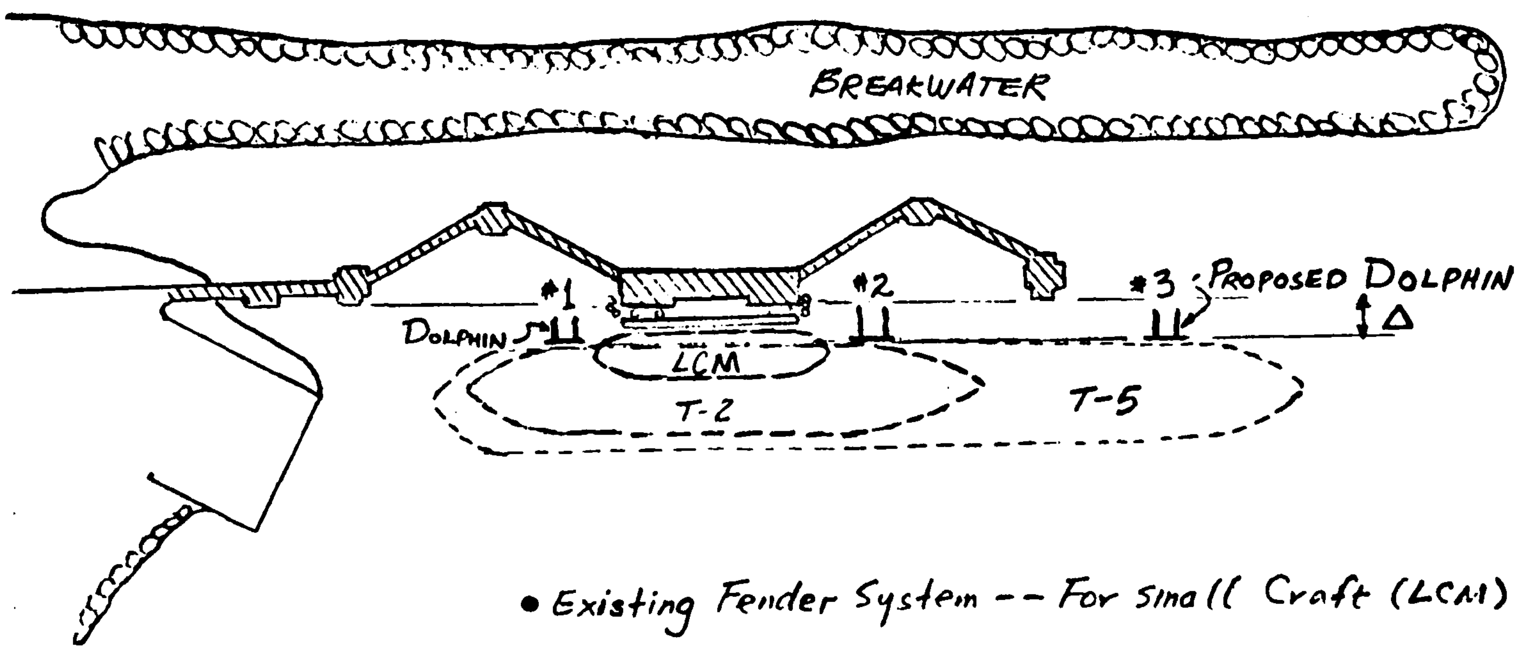


Figure A.1 PROPOSED FENDER SYSTEM IMPROVEMENT

ATTACHMENT (2)

2-2-78
FPO-1EA21

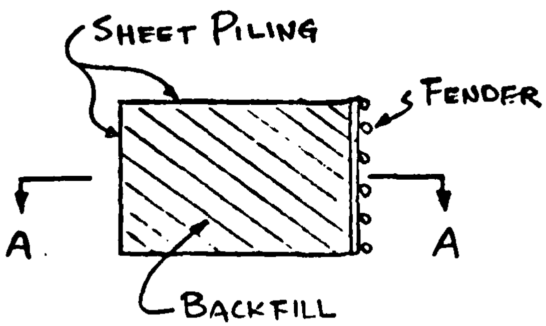
Figure A.2 SCHEME No. I POL PIER UP TO T-5 CLASS



- Existing Fender System -- For Small Craft (LCM)
- Proposed Loading Dolphin System
  - #1 & #2 -- For T-2 Class
  - #1, #2 & #3 -- For T-5 Class
- Req. some dredging for T-5 class

ATTACHMENT (3)

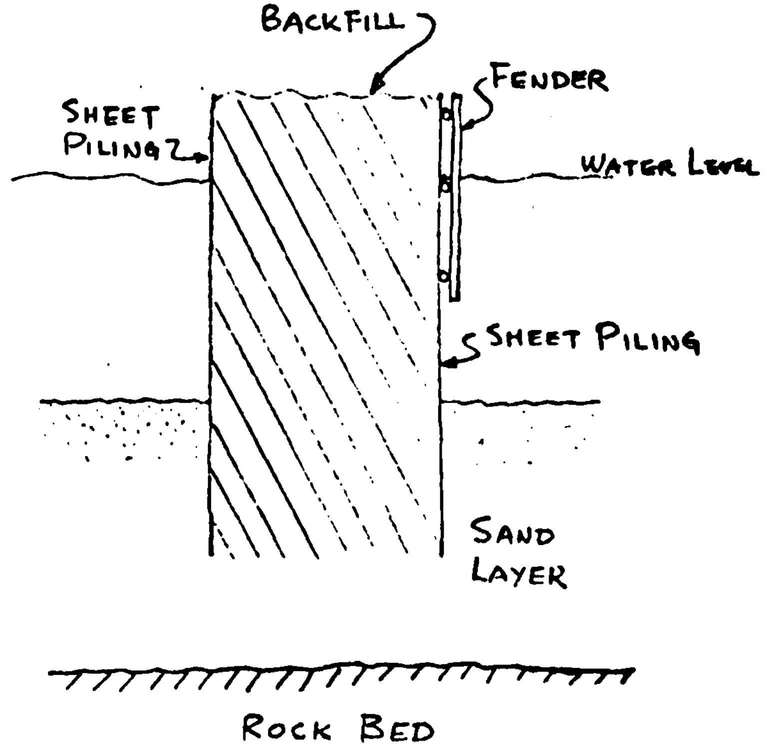
2-2-78
FPO-1EA21



PLAN

- STEEL SHEET PILINGS
  W/GRAVEL & SAND BACKFILL

or
- CONCRETE CAISSON
  W/GRAVEL & SAND BACKFILL



VIEW A-A

Figure A-3 FRICTION RESISTANCE TYPE DOLPHINS

ATTACHMENT (4)

- STEEL PIPE PILES
  OR H-PILES
- REQUIRE DRILLING INTO
  BED ROCK
- ELIMINATE TENSION ON
  VERTICAL PILES
  -- ENERGY ABSORBING MECHANISM

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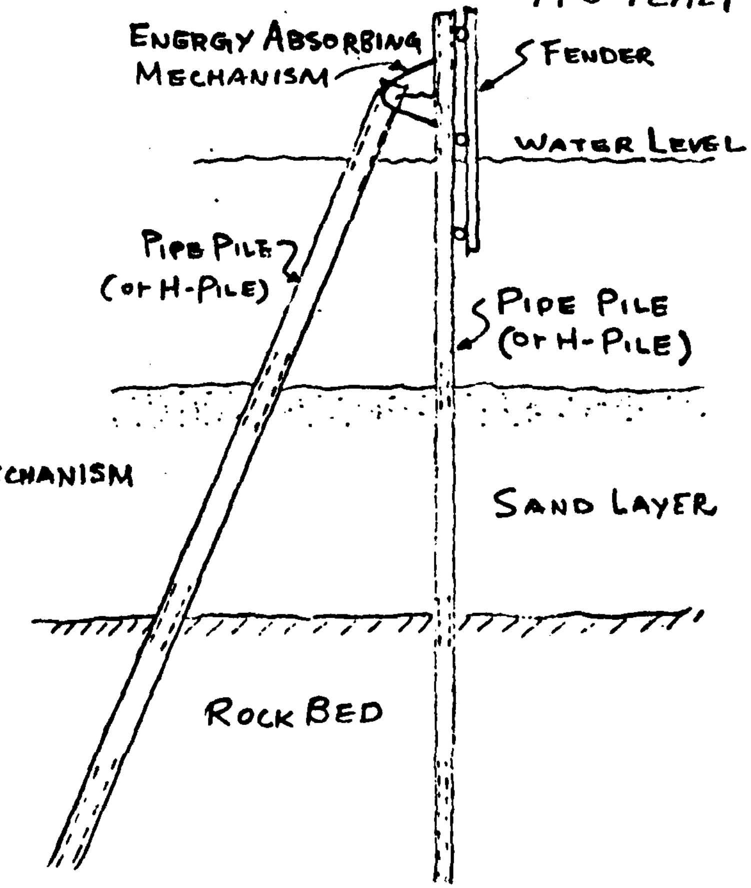
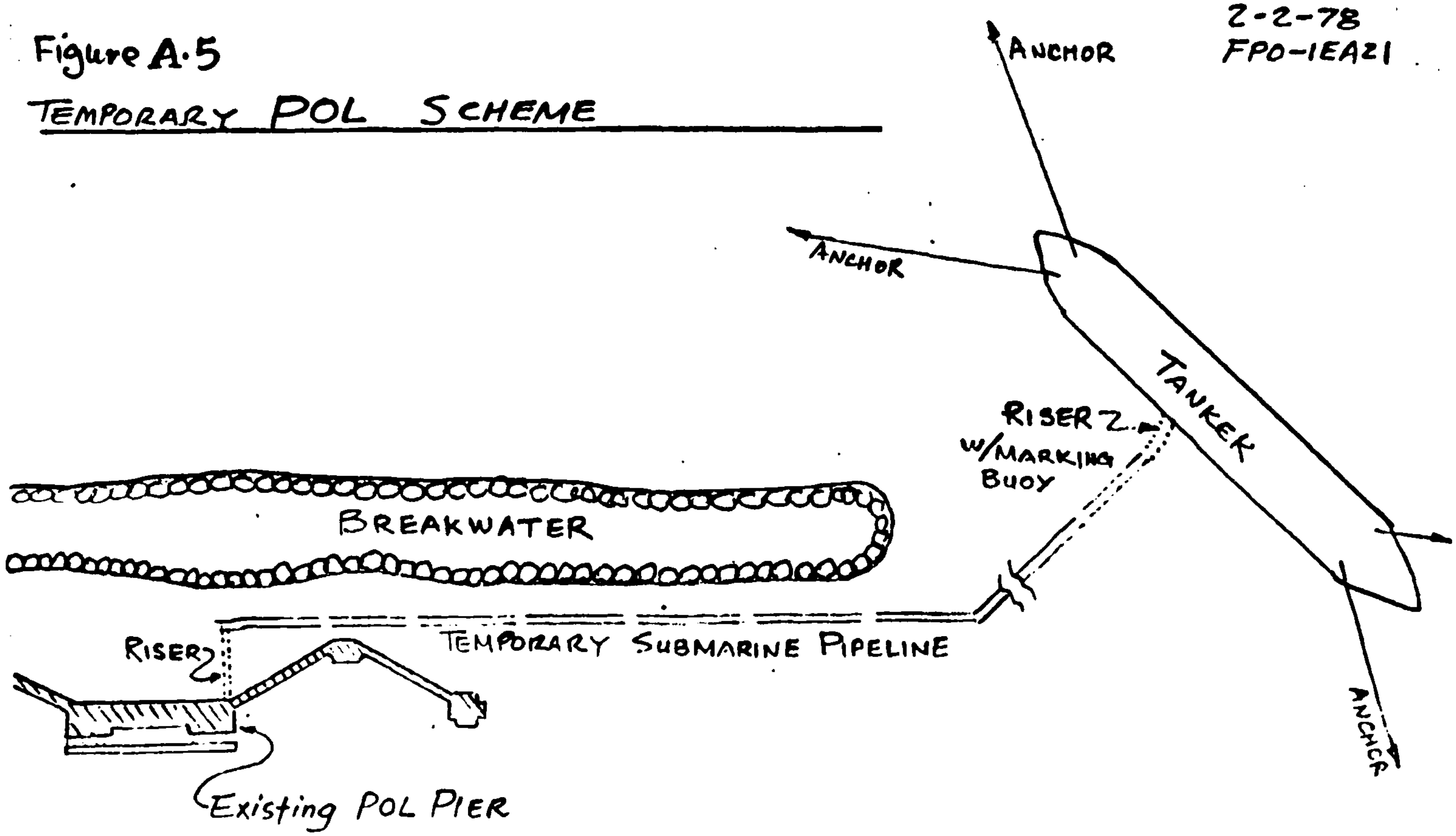


Figure A4 ENERGY ABSORPTION TYPE DOLPHINS

ATTACHMENT (5)

Figure A.5
TEMPORARY POL SCHEME

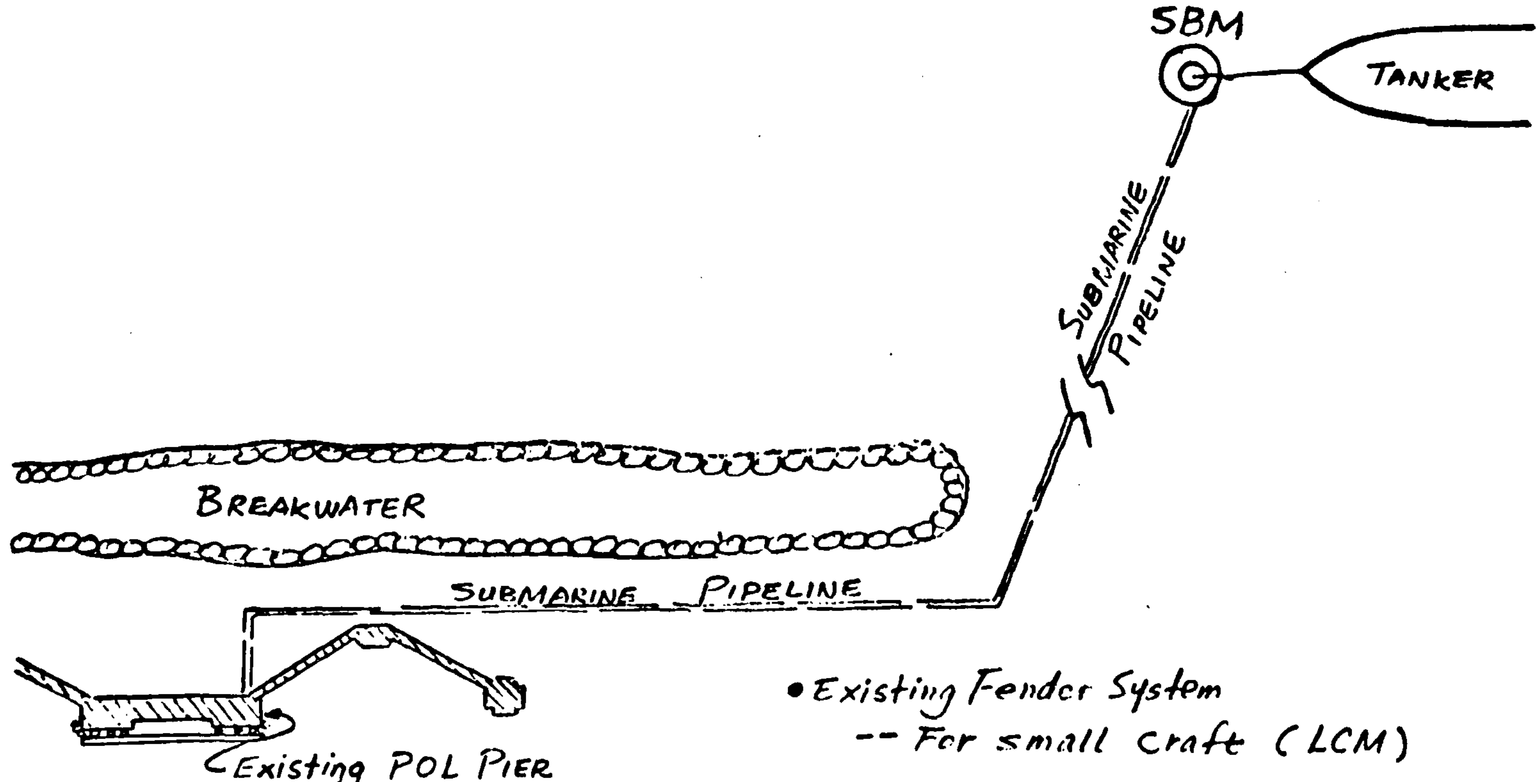
2-2-78
FPO-1EA21



ATTACHMENT (6)

2-2-78
FPO-1EA21

Figure A.6 SCHEME No.2 SBM SYSTEM FOR TANKER UP TO 200,000 DWT



- Existing Fender System
  -- For small craft (LCM)
- SBM System
  -- For oil tanker only

APPENDIX D

POL Pier Structural Evaluation

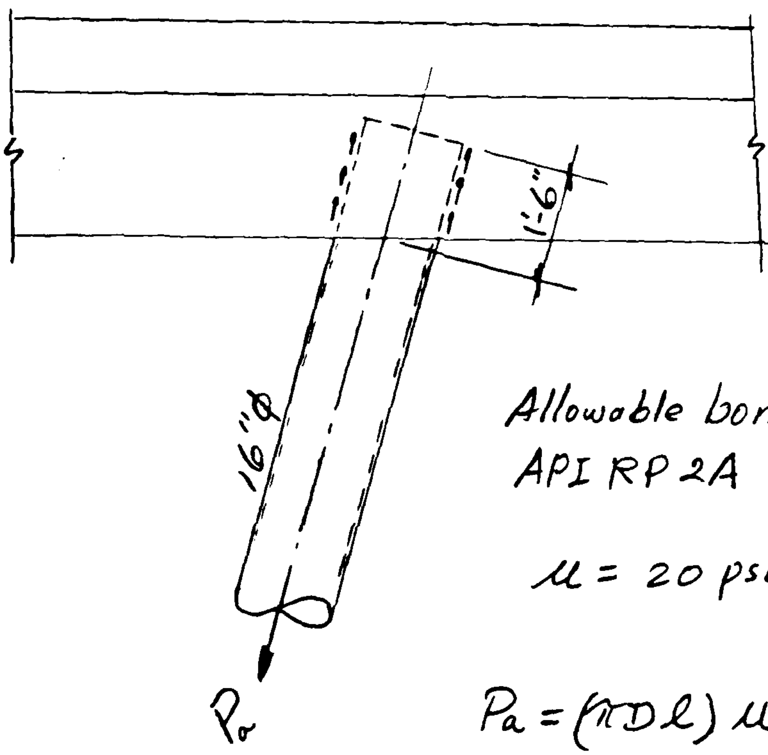
Th is section compiles the engineering calculations on the structural components related to the ultimate strength of the existing pier structure and the operating limitations of the proposed new fender system.

D.1 Pile Cap Pull-Out Strength

In the absence of reliable data on the bond stress between the pile surface and the concrete cap, the allowable stress of 20 psi from reference 4 is used to compute the pile allowable pull-out force. The ultimate pull-out force is then obtained by multiplying a factor of safety of 2.0 to the allowable pull-out force. It is noted that the factor of safety of 2.0 is selected by common practice under such circumstance (Ref.4).

C. Chern
2-13-78

BOND BETWEEN STEEL PILE AND CONCRETE CAP



Allowable bond stress per API RP 2A §2.36a (8^{th} Ed.)

$u = 20$ psi

$$P_a = (\pi D l)\, u$$

where P_a = allowable tensile force

D = pile diameter

l = embedment length

u = bond stress

C. Chern
2-13-78

$$P_a = \pi \times 16 \times 18 \times 20 = 18{,}096 \text{ lbs}$$

$$P_u = (F.S.) \times P_a$$

where P_u = ultimate pull-out force

$F.S.$ = factor of safety, 2.0 for this case

$$P_u = 2 \times 18{,}096$$
$$= 36{,}192 \text{ lbs}$$

AD-A165 765 LAJES FIELD AZORES POL PIER FIELD INVESTIGATIONS AND RECOMMENDATIONS(U) NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON DC CHESAPEAKE.. C CHERN ET AL. MAR 78 2/2

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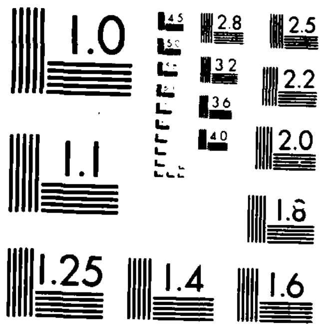


MICROCOPY RESOLUTION TEST CHART

D.2 Pile Foundation Pull-out Strength

The ultimate pull-out strength of the 16" Ø steel pipe pile in sand is computed in this section. In the computations, it is assumed that the pile pull-out resistance is contributed solely from the friction between the pile embeded surface and the neighboring sand.

Clean sand characteristics are used in the computations. Pile penetration is 40 feet below mud-line.

C. Chern
2-13-78

PILE CAPACITY FOR AXIAL PULLOUT LOADS in Sand

(Ref: API RP 2A §2.28 8th Edition)

$$Q_u = f A_s$$

where Q_u = ultimate pile pullout capacity, lbs

f = unit skin friction capacity, lbs/ft²

A_s = side surface area of pile, ft²

Friction in Sand

$$f = K\, p_o \tan\phi'$$

where K = coefficient of lateral earth pressure, 0.5 for pile in tension

p_o = effective overburden pressure, lb/ft²

ϕ' = angle of soil friction on pile wall, degree, 30° for clean sand

C. Chern
2-13-78

For 16"ϕ pile $\quad A_s = \pi D h = \pi \times \frac{16}{12} \times 5 = 20.94 \ \square'$

$(h = 5 \text{ ft})$

γ' = submerged weight of sand
65 #/cu.ft

$\phi' = 30^\circ \quad p_o = \gamma' h = 65 h$

$\tan\phi' = 0.577 \quad K = 0.5$

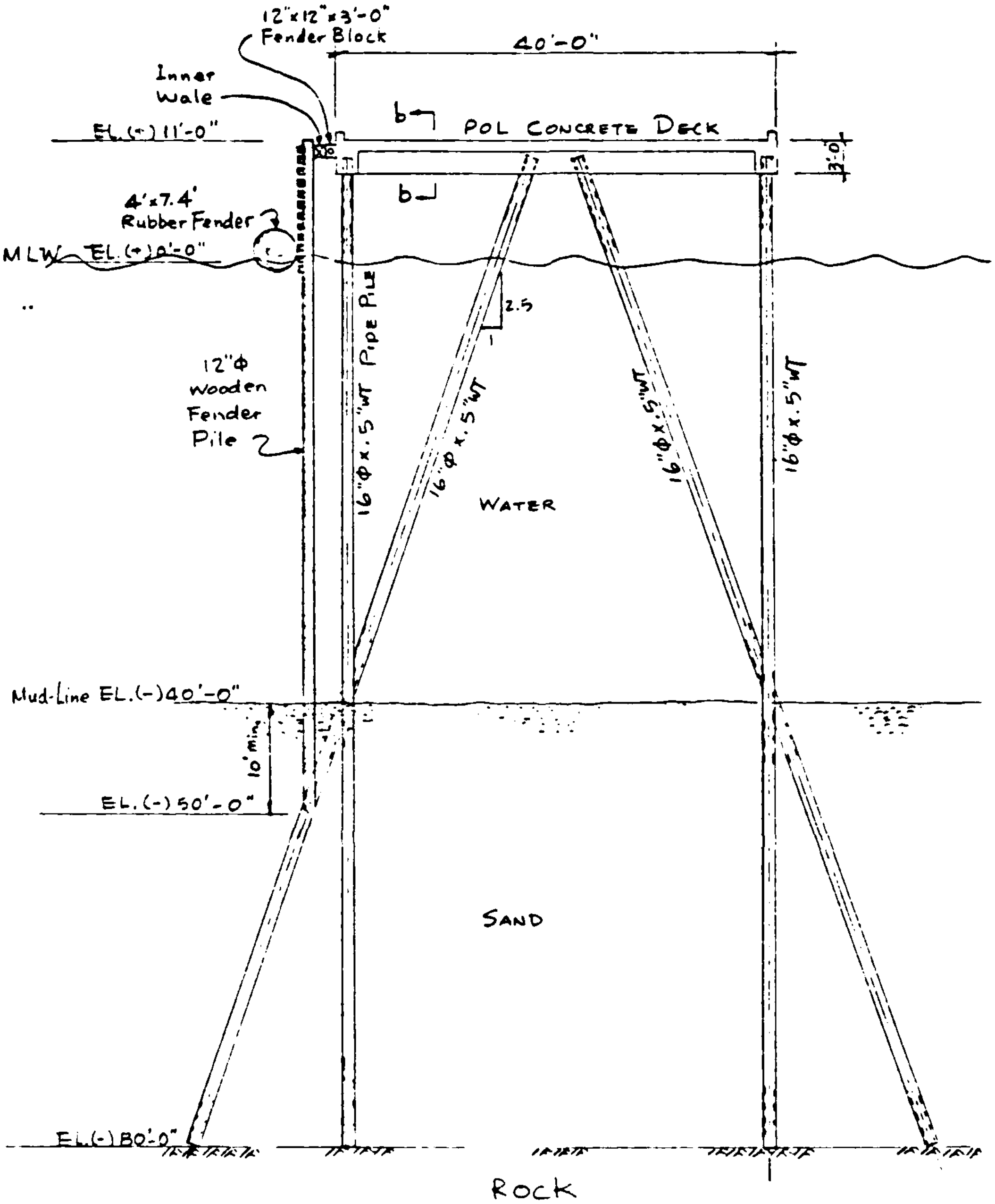
PENETRATION FT	$f = K p_o \tan\phi'$ #/□'	f_{ave} #/□'	$\Delta Q_u = f_{ave} \cdot A_s$ Lbs	$Q_u = \Sigma \Delta Q_u$ Lbs
0 5	0 94	47	984	0 984
5 10	187.5	141	2,953	984 3,937
10 15	281	234	4,900	3,937 8,837
15 20	375	328	6,868	8,837 15,705
20 25	469	422	8,837	15,705 24,542
25 30	563	516	10,805	24,542 35,347
30 35	656	610	12,773	35,347 48,120
35 40	750	703	14,721	48,120 62,841

D.3 POL Pier Structural Strength

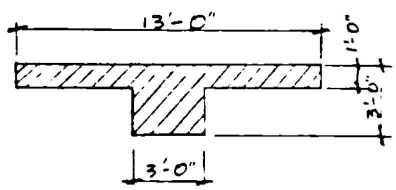
The structural component failures and the corresponding lateral load resisting capacities of the pier are evaluated in this section. The possible structural component failures are:

- pile cap pull-out
- pile foundation pull-out
- concrete frame beam failure by tension yielding.

The lateral load causing the pile cap pull-out is found to be the lowest failure load of the pier structure (27 kips per loading frame). Subsequent increase in the lateral load application will cause the concrete frame beam to fail at the mid-span location by tension yielding of the top surface reinforcing bars. The ultimate lateral load resistance per frame is approximately at 39 kips.



(a) POL Pier Cross-Section



(b) View b-b Cross-Section

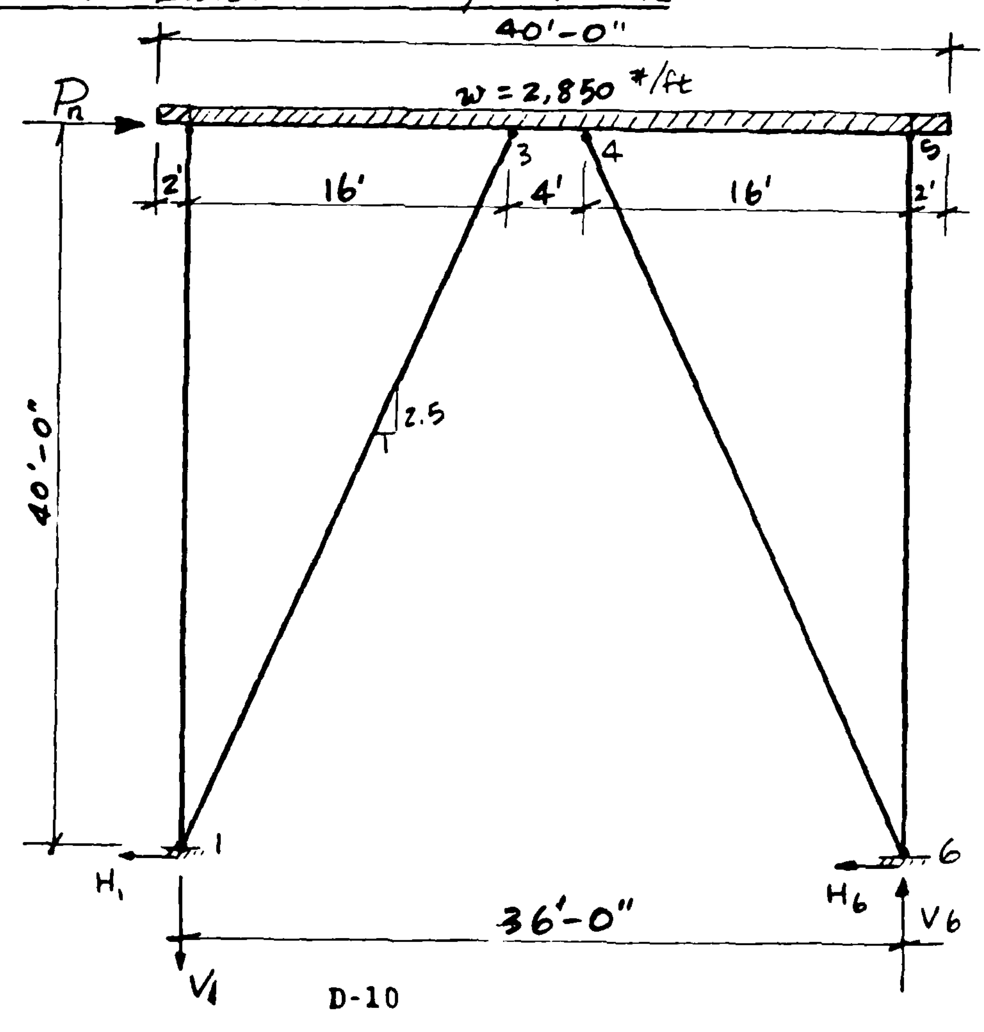
C. Chern
2-14-78

NORTH BENT

D.W. of cross-section (View 6-6)

$$w = 150 \times [13 \times 1 + 2 \times 3] = 2{,}850 \ \#/ft$$

Ultimate Lateral Load per Frame



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$\Sigma M_6 = 0 \qquad V_1 \times 36 + 2,850 \times 40 \times 18 = P_n \times 40$

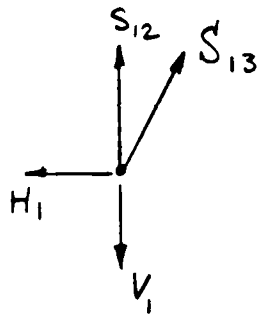
$$V_1 = \frac{10}{9} P_n - 57,000 \qquad (Lbs)$$

$$V_6 = V_1 + 2,850 \times 40$$

$\Sigma F_h = 0 \qquad H_1 + H_6 = P_n$

$$H_1 = H_6 = \frac{1}{2} P_n$$

@ Joint 1



$$S_{13} \cdot \frac{1}{\sqrt{7.25}} = H_1$$

$$S_{13} = \sqrt{7.25}\, H_1 = 1.346 P_n$$

$$V_{13} = 2.5 H_1 = 1.25 P_n$$

$$S_{12} + V_{13} = V_1$$

$$S_{12} = V_1 - V_{13}$$

$$= \frac{10}{9} P_n - 57,000 - 1.25 P_n < 0$$

(Under compression)

Bond stress @ Joint #3 on Member 1-3

$$(S_{13})_{allowable} = 18,096 \text{ Lbs}$$

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Say F.S. = 2.0 for bond stress @ 20 psi

Ultimate pull-out force

$$(S_{13})_u = 2 \times 18{,}096$$

$$= 36{,}192 \text{ lbs}$$

$$1.346 P_n = 36{,}192$$

$$P_n = 26{,}888 \text{ lbs}$$

Say $\underline{\underline{P_n = 27{,}000 \text{ lbs}}}$

$$V_1 = \frac{10}{9} \times 27{,}000 - 57{,}000 = -27{,}000 \text{ lbs} \uparrow \text{ (comp.)}$$

$$H_1 = \frac{1}{2} P_n = 13{,}500 \text{ lbs} \rightarrow$$

$$H_6 = 13{,}500 \text{ lbs} \rightarrow$$

$$V_6 = -27{,}000 + 114{,}000 = 87{,}000 \text{ lbs} \uparrow \text{ (Comp.)}$$

$$S_{12} = \frac{10}{9} \times (+27{,}000) - 57{,}000 - 1.25(+27{,}000)$$

$$= -60{,}750 \text{ lbs (comp)}$$

$$S_{13} = 1.346 P_n = 36{,}342 \text{ lbs (tens.)}$$

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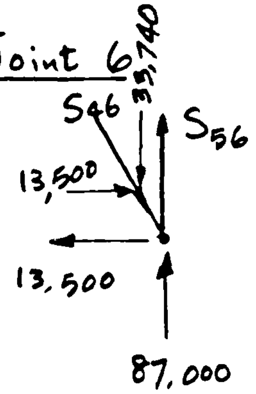
$$S_{46} = |S_{13}| = 36,342 \text{ lbs (comp.)}$$

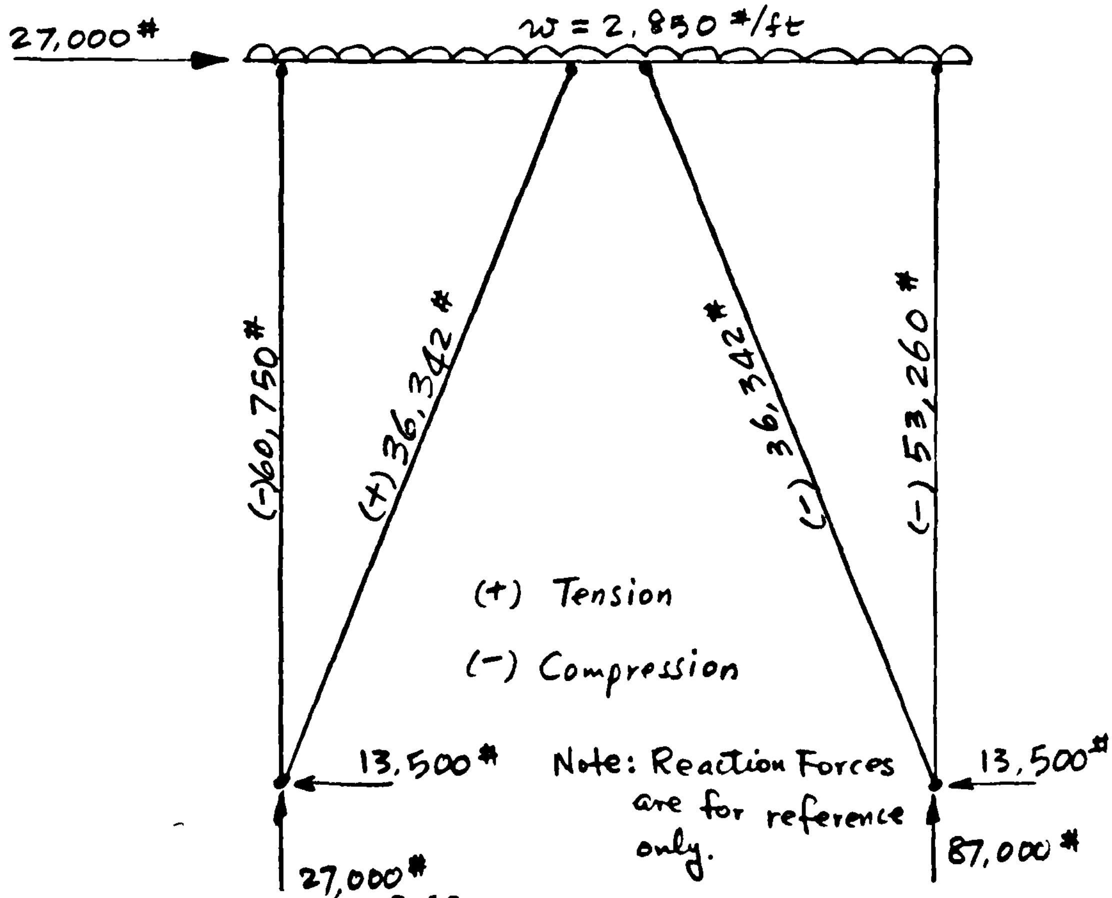
<u>Joint 6</u>



$$\Sigma F_v = 0$$

$$S_{56} + 87,000 = 33,740$$

$$S_{56} = -53,260 \text{ lbs (Comp.)}$$



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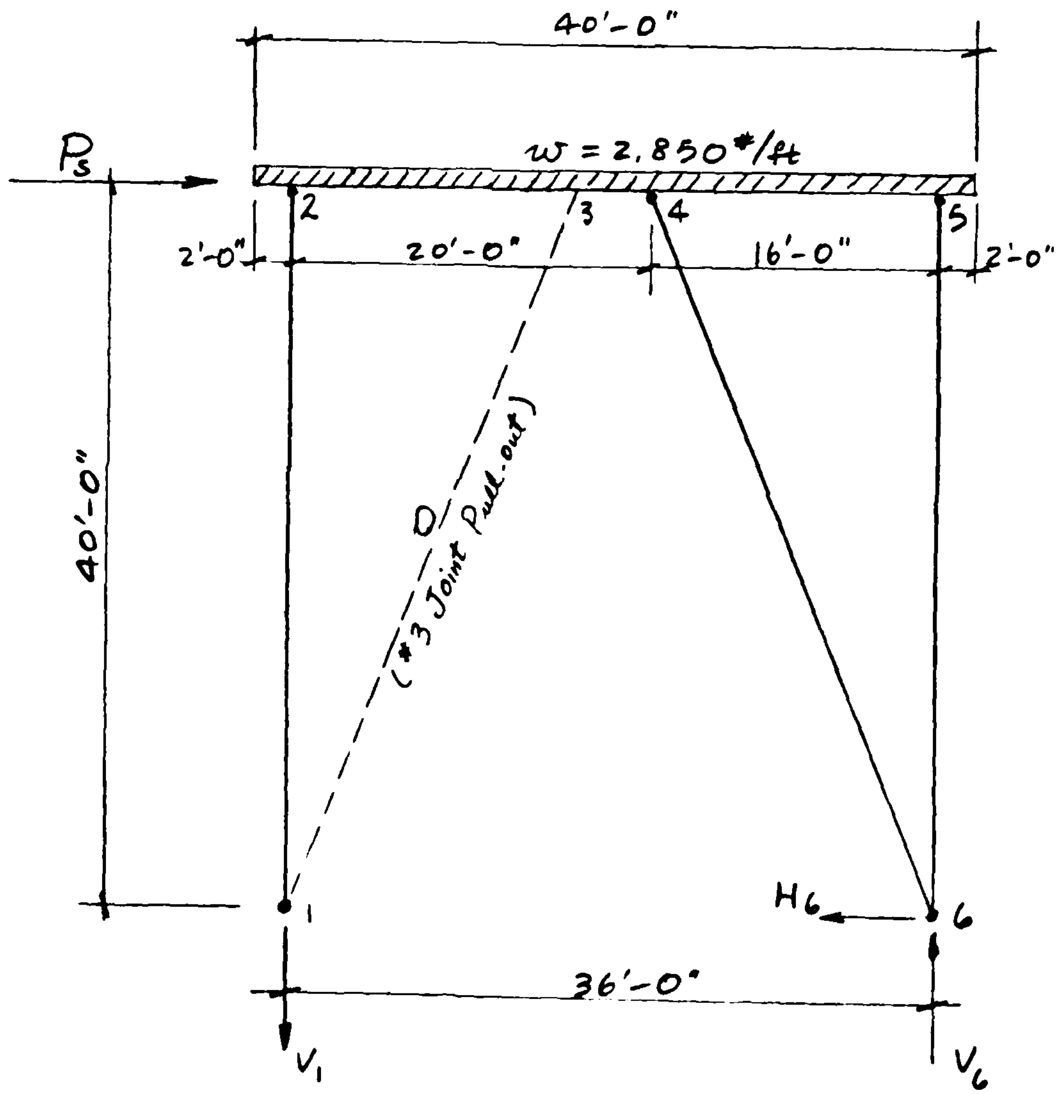
SOUTH BENT



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$\Sigma M_6 = 0$

$$V_1 \times 36 + 2{,}850 \times 40 \times 18 = P_s \times 40$$

$$V_1 = \frac{10}{9} P_s - 57{,}000$$

$$S_{12} = V_1$$

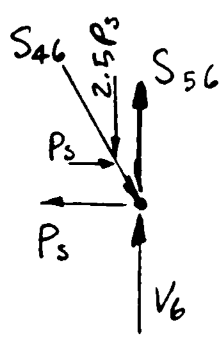
$$S_{46} = \sqrt{7.25}\, P_s$$

$$V_6 = V_1 + 2{,}850 \times 40$$

$$= \frac{10}{9} P_s - 57{,}000 + 114{,}000$$

$$= \frac{10}{9} P_s + 57{,}000$$

Joint 6



$$S_{56} = 2.5P - V_6$$

$$= 2.5P_s - \frac{10}{9} P_s - 57{,}000$$

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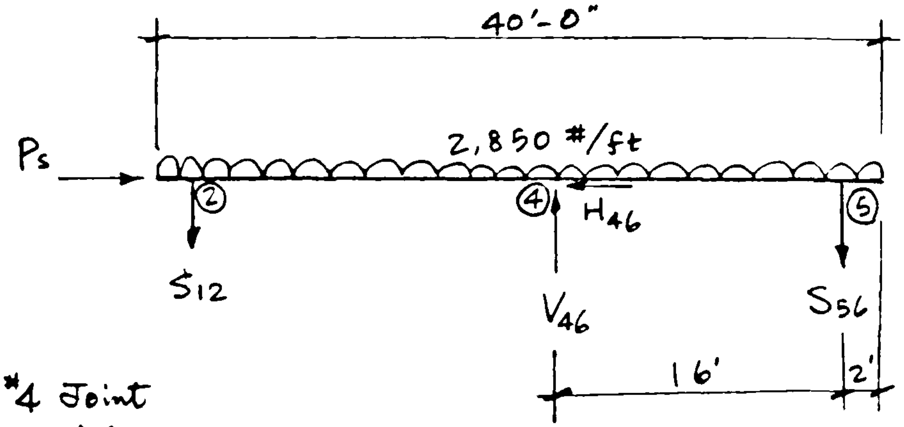

@ #4 Joint

$$M_4 = 2,850 \times 18 \times 9 + S_{56} \times 16$$

$$= 461,700 + (2.5 P_s - \frac{10}{9} P_s - 57,000) \times 16$$

$$= \frac{200}{9} P_s - 450,300 \quad (\text{ft-Lbs})$$

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Case 1 S_{12} pullout

$$36,192 = \frac{10}{9} P_s - 57,000$$

$$P_s = 83,873 \text{ lbs}$$

Case 2 S_{56} pullout

$$36,192 = 1.39 P_s - 57,000$$

$$P_s = 67,045 \text{ lbs}$$

Say $P_s = 67,000$ lbs

$$S_{12} = \frac{10}{9} \times 67,000 - 57,000 = +17,444 \text{ lbs}$$
(Tens.)

$$S_{46} = \sqrt{7.25}\, P_s = 180,403 \text{ lbs (comp.)}$$

$$S_{56} = 36,192 \text{ lbs (Tens.)}$$

Case 3 Beam Bending Failure @ #4 Joint

$$M = \frac{200}{9} P_s - 450,300 \quad \text{(ft-lbs)}$$

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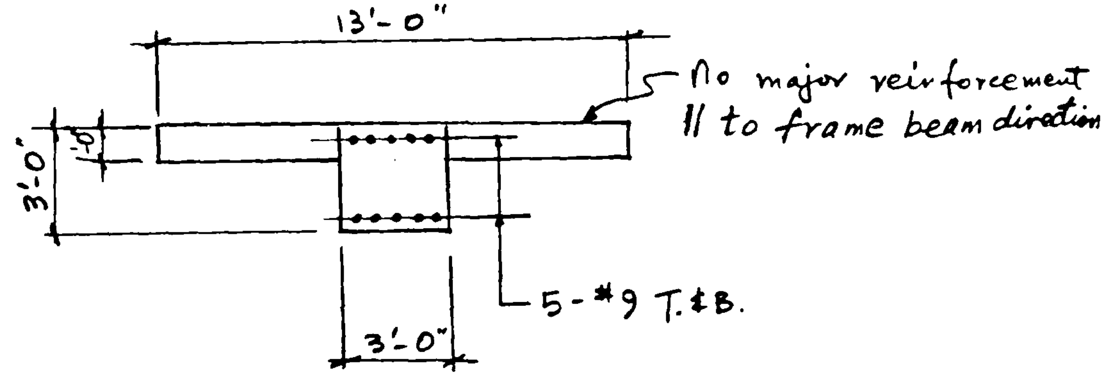
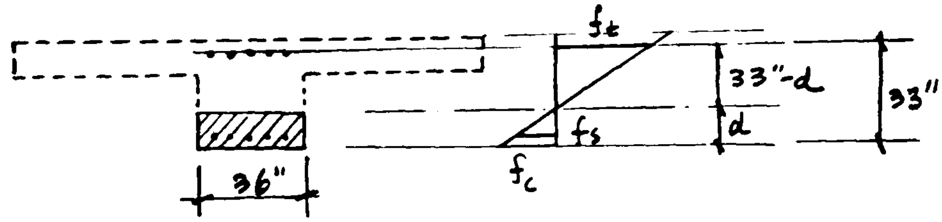
T-SECTION UNDER NEGATIVE MOMENT



Effective section for negative moment resistance



$$n = \frac{E_s}{E_c} = 10$$

$$F_t = f_t \cdot n A_s = f_t \cdot (10 \times 5 \times 1) = 50 f_t$$

$$F_c = F_{c_1} + F_{c_2}$$

$$F_{c_1} = \frac{1}{2} f_c \cdot \times 36 \times d = 18 d f_c$$

$$F_{c_2} = n A_s \cdot f_s = (10 \times 5 \times 1) \cdot f_s = 50 f_s$$

$f_s : f_c = (d-3) : d \qquad f_s = (1 - 3/d) f_c$

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$$F_c = 18d\,f_c + 50\left(1-\frac{3}{d}\right)f_c$$

$\Sigma F = 0 \qquad F_t = F_c$

$$50 f_t = \left(18d + 50 - \frac{150}{d}\right) f_c$$

$$f_t = \left(\frac{9}{25}d + 1 - \frac{3}{d}\right) f_c \quad \cdots\cdots \quad (a)$$

$$f_t : f_c = (33-d) : d$$

$$f_t = \left(\frac{33}{d} - 1\right) f_c \quad \cdots\cdots\cdots \quad (b)$$

Substitute (b) into (a)

$$\left(\frac{33}{d} - 1\right)\cancel{f_c} = \left(\frac{9}{25}d + 1 - \frac{3}{d}\right)\cancel{f_c}$$

$$\frac{9}{25}d + 2 - \frac{36}{d} = 0$$

$$9d^2 + 50d - 900 = 0$$

$$d = \frac{-50 + \sqrt{50^2 + 4\times 9 \times 900}}{2\times 9}$$

$$d = 7.6''$$

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Let $f_t = \frac{f_y}{n} = \frac{36,000}{10} = 3,600$ psi

from (b) $f_c = \frac{f_t}{\frac{33}{d} - 1} = \frac{3,600}{\frac{33}{7.6} - 1} = 1,077$ psi

$< f_c' = 3,000$ psi O.K.

$f_s = (1 - \frac{3}{d}) f_c$

(File No. 7571-5799 Appendix B-1)

$= (1 - \frac{3}{7.6}) \times 1,077$

$= 652$ psi

$F_t = 50 \times 3,600 = 180,000$ lbs

$F_c = 18 \times 7.6 \times 1,077 + 50 \times 652 = 180,000$ lbs

$M = F_t \cdot (33 - d) + F_{c_1} \times \frac{d}{3} + F_{c_2} \times (d - 3)$

$= 180,000 \times (33 - 7.6) + 147,333 \times \frac{7.6}{3}$

$+ 32,667 \times (7.6 - 3)$

$= 5,095,512$ "#

$\underline{\underline{M = 424.6^{ft\text{-}k}}}$

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$$M = \frac{200}{9} P_s - 450,000 \quad (ft\text{-}lbs)$$

$$424,600 = \frac{200}{9} P_s - 450,000$$

$$\frac{200}{9} P_s = 39,357 \text{ lbs}$$

$$S_{12} = \frac{10}{9} \times 39,357 - 57,000$$

$$= -13,270 \text{ lbs} \quad (\text{comp.})$$

$$S_{46} = \sqrt{7.25} \times 39,357$$

$$= 105,972 \text{ lbs} \quad (\text{comp.})$$

$$S_{56} = 2.5 \times 39,357 - \frac{10}{9} \times 39,357 - 57,000$$

$$= -2,338 \text{ lbs} \quad (\text{comp.})$$

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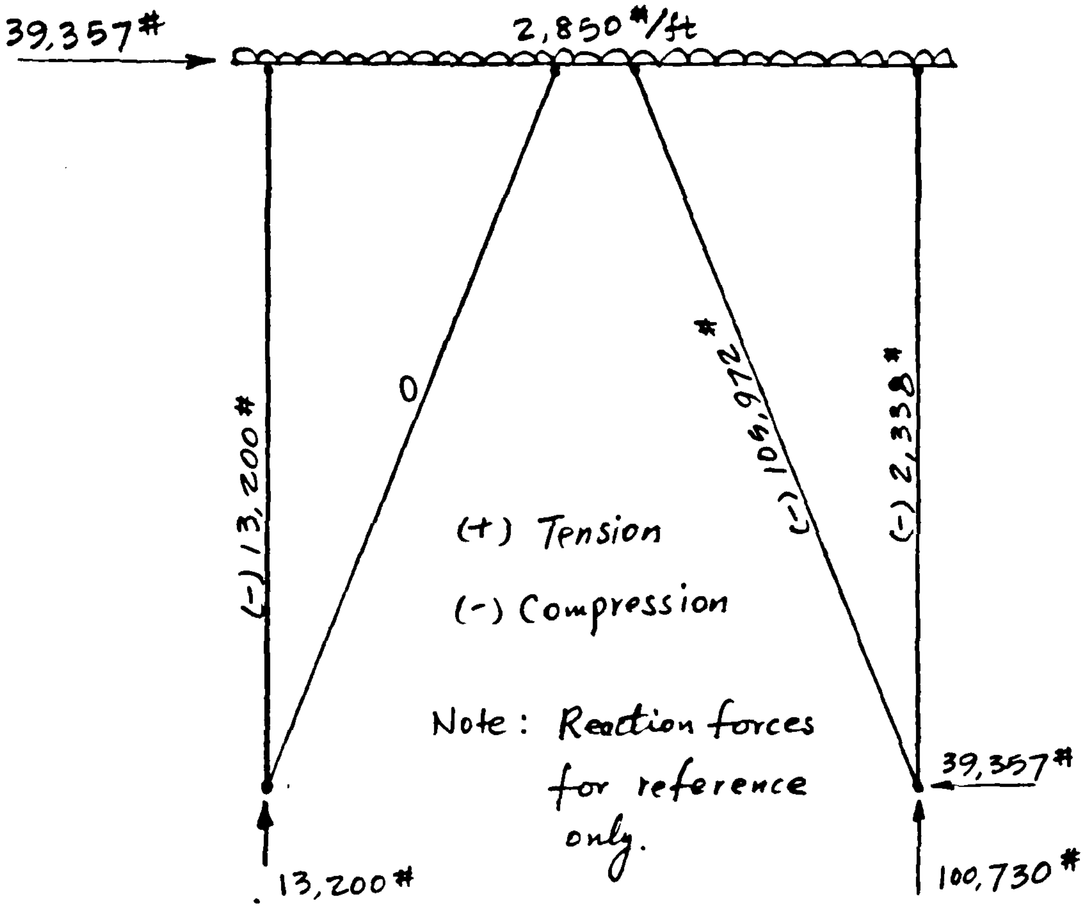

D.4 Berthing Energy and Transferred Force

The computations herein are performed on the assumption that the 4' Ø x 7.4' ℒ SEA CUSHION Marine Fender will be used in the proposed new fender system. It is estimated that four 4' Ø x 7.4' ℒ SEA CUSHION shall be used in the system

The absorbed berthing energies and the corresponding transferred forces are computed, respectively, for the water levels at EL.(-)3'-0", (-)2'-0", (-)1'-0", (+)0'-0" MLW, (+)1'-0", (+)2'-0" and (+)3'-0".

The allowable berthing force and energy are limited by the strength of either the fender piles or the pier frame structure. The criteria for the fender pile strength are:

- Class II treated southern pine or douglas fir
- limiting flexural stress of the extreme fiber is at 1,750 psi
- the pile cross section at loading point will have at least 18" in diameter.

The pier structural strength is used later in §D.7.

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BERTHING FORCE AND ENERGY COMPUTATIONS

[Ref. 6: SEA CUSHION MARINE FENDERS
Technical Manual SCTM-4 (5-77)]

Ship Berthing Energy Calculations

T-2 Class

W_b, Displacement Tonnage (L.T.) = 21,800

D, Draft (ft) = 31

L, Length (ft) = 501.4

W_a, Added Mass $= \frac{\pi}{4} \rho D^2 L$ (L.T.) = 10,813

($\rho = 64$ #/cu.ft)

$W = W_a + W_b$ (L.T.) = 32,613

V, Relative Velocity (ft/sec) = 0.25

C_B, Berthing Coefficient = 0.5

g, Acceleration of Gravity (ft/sec²) = 32.2

$$E = C_B \cdot \frac{W V^2}{2g}$$

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$$E = 0.5 \times \frac{32{,}613 \times (0.25)^2}{2 \times 32.2}$$

$= 15.8$ ft - L.T.

$= 35.4$ ft - kips

Use Sea Cushion ($4'\phi \times 7.4'\ell$) @ 60% Compression

Energy Absorption = 56.7 ft-kips

Reaction Force = 64 kips

For berthing energy @ $E = 35.4$ ft-kips

$4'\phi \times 7.4'\ell$ SEA Cushion

48% compression

Reaction Force = 40 kips

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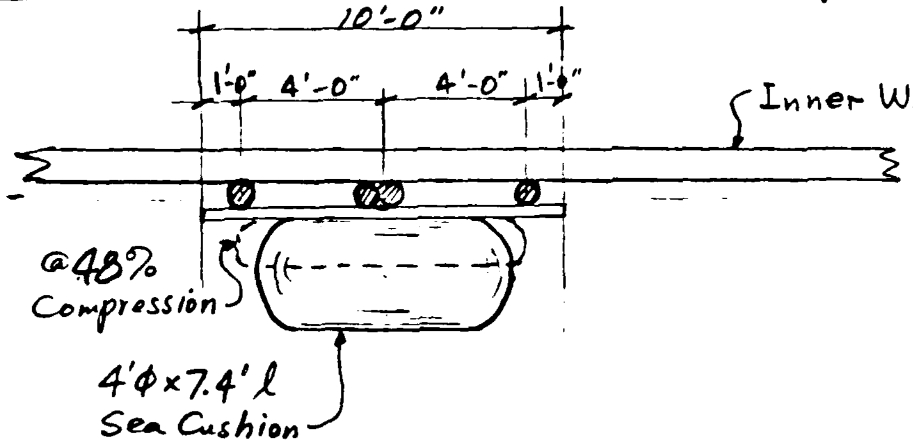
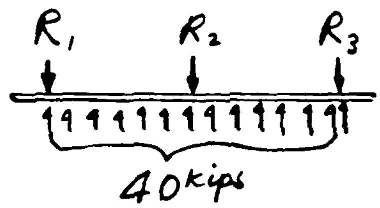
4'Φ × 7.4l Sea Cushion @48% Compression





$R_1 = R_3$

$R_2 = R_1 + R_3$

$R_2 = \frac{40}{2} = 20.0$ kips

* Center pile (or piles) shall carry 20.0 kips transferred force.

Further assumed that the berthing force (2F) will be distributed in the following proportions:

$R_2 = F$

$R_1 = R_3 = \frac{1}{2} F$

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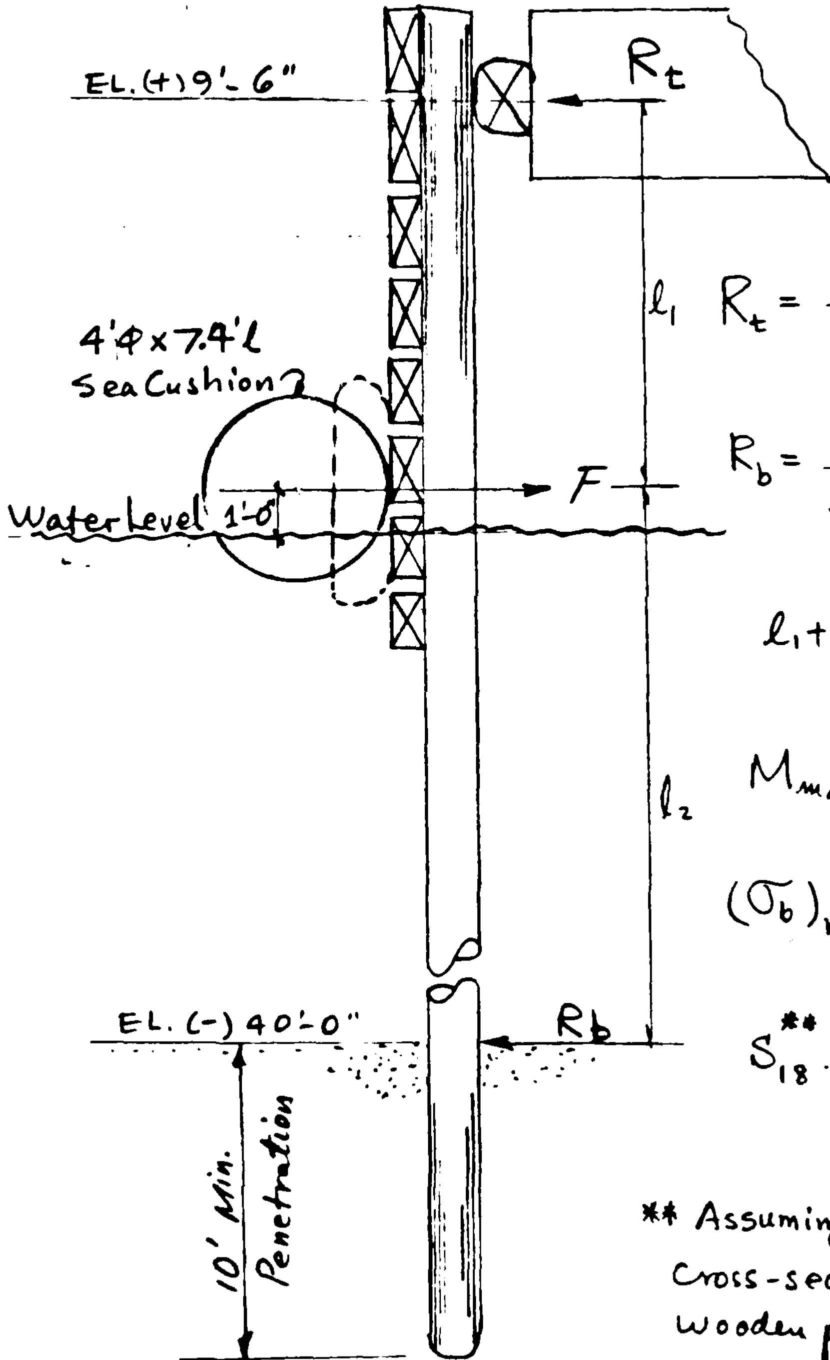
Assuming that inner wale will deflect under load and will have continuous contact with pier.



$$R_t = \frac{l_2}{l_1+l_2} F$$

$$R_b = \frac{l_1}{l_1+l_2} F$$

$$l_1 + l_2 = 49.5'$$

$$M_{max} = R_t l_1$$

$$(\sigma_b)_{max} = \frac{M_{max}}{2 \cdot S_{18}}$$

$$S_{18}^{**} = \frac{\pi}{32} \times 18^3$$
$$= 572.6 \text{ in}^3$$

** Assuming that effective cross-section of 12"Φ nominal wooden pile @ loading point will have 18"Φ equivalent

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Water Level @ M.L.W. EL.(+)0'-0"

$$l_1 = 9.5 - 1 = 8.5'$$

$$R_t = \frac{41}{49.5} F = 0.83F$$

For dense select structural douglas fir $(\sigma_b)_{limiting} = 1{,}750$ psi

(Ref.7 Wood Engineering by G. Gurfinkel)

$$1{,}750 = \frac{0.83F \times 8.5 \times 12}{2 \times 572.6}$$

$$F = 23{,}672^{\#}$$

$$R_t = 19{,}648^{\#}$$

$$R_b = 4{,}024^{\#}$$

$$\text{Berthing Force} = 2F = 47{,}344^{\#} = 47.3 \text{ kips}$$

4'ϕ × 7.4'l Cushion @ 53% compression

Berthing Energy = 42.5 ft-kips

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Water Level @ EL.(+) 1'-0"

$$\ell_1 = 9.5 - (1+1) = 7.5'$$

$$R_t = \frac{42}{49.5} F = 0.85F$$

$$M_{max} = R_t \ell_1$$

$$= 0.85F \times 7.5 \times 12$$

$$= 76.5\,F \quad \text{in-lbs}$$

$$1.750 = \frac{76.5F}{2 \times 572.6}$$

$$F = 26{,}197^{\#}$$

$$R_t = 22{,}267^{\#}$$

$$R_b = 3{,}930^{\#}$$

Berthing Force $= 2F = 52{,}394^{\#}$

$= 52.4$ kips

4'ϕ x 7.4'ℓ Cushion @ 56% Compression

Berthing Energy = 47 ft-kips

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Water Level @ EL.(+) 2'-0"

$$l_1 = 9.5 - (1+2) = 6.5'$$

$$R_t = \frac{43}{49.5} F = 0.87F$$

$$M_{max} = R_t l_1$$

$$= 0.87F \times 6.5 \times 12$$

$$= 67.86F \quad \text{in-Lbs}$$

$$1{,}750 = \frac{67.86F}{2 \times 572.6}$$

$$F = 29{,}533^{\#}$$

$$R_t = 25{,}694^{\#}$$

$$R_b = 3{,}839^{\#}$$

Berthing Force $= 2F = 59{,}066^{\#}$
$= 59$ kips

4'ϕ × 7.4'l Cushion @ 58 % compression

Berthing Energy = 59 ft-kips

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Water Level @ EL.(+)3'-0"

$$l_1 = 9.5' - (1+3) = 5.5'$$

$$R_t = \frac{44}{49.5} F = 0.89 F$$

$$M_{max} = R_t \, l_1$$

$$= 0.89 F \times 5.5 \times 12$$

$$= 57.42 F \quad \text{in-lbs}$$

$$1,750 = \frac{57.42 F}{2 \times 572.6}$$

$$F = 34,902 \#$$

$$R_t = 31,063 \#$$

$$R_b = 3,839 \#$$

$$\text{Berthing Force} = 2F = 69,804 \#$$

$$= 69.8 \text{ kips}$$

4'ϕ x 7.4'l Cushion @ 62% Compression

Berthing Energy = 65 ft-kips

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Water Level @ EL.(-) 1'-0"

$\ell_1 = 9.5' - (1-1) = 9.5'$

$R_t = \frac{40}{49.5} F = 0.81 F$

$M_{max} = R_t \ell_1$

$= 0.81 F \times 9.5 \times 12$

$= 92.34 F$ in-lbs

$1.750 = \frac{92.34 F}{2 \times 572.6}$

$F = 21,703$ #

$R_t = 17,579$ #

$R_b = 4,124$ #

Berthing Force = 2F = 43,406 #
= 43.4 kips

4'ϕ × 7.4'ℓ Cushion @ 51% compression

Berthing Energy = 37 ft-kips

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Water Level @ EL.(-) 2'-0"

$$l_1 = 9.5' - (1-2) = 10.5'$$

$$R_t = \frac{39}{49.5} F = 0.79 F$$

$$M_{max} = R_t \, l_1$$

$$= 0.79 F \times 10.5 \times 12$$

$$= 99.54 F \quad \text{in-lbs}$$

$$1{,}750 = \frac{99.54 F}{2 \times 572.6}$$

$$F = 20{,}134 \#$$

$$R_t = 15{,}906 \#$$

$$R_b = 4{,}228 \#$$

Berthing Force = 2F = 40,268#
= 40.3 kips

4'Φ × 7.4'ℓ Cushion @ 48% Compression

Berthing Energy = 33 ft-kips

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<u>Water Level @ EL.(-)3'-0"</u>

$$\ell_1 = 9.5' - (1-3) = 11.5'$$

$$R_t = \frac{38}{49.5} F = 0.77 F$$

$$M_{max} = R_t \ell_1$$

$$= 0.77 F \times 11.5 \times 12$$

$$= 106.26 F$$

$$1.750 = \frac{106.26 F}{2 \times 572.6}$$

$$F = 18{,}860^{\#}$$

$$R_t = 14{,}522^{\#}$$

$$R_b = 4{,}338^{\#}$$

Berthing Force $= 2F = 37{,}720^{\#}$
$= 37.7$ kips

4'ϕ × 7.4'ℓ Cushion @ 45% Compression

Berthing Energy = 26 ft-kips

D.5 Pile Lateral Bearing Capacity in Sand

The lateral bearing capacity of the 12" Ø nominal wooden pile is evaluated in this section. The results will provide information on the fender pile penetration requirements.

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PILE LATERAL BEARING CAPACITY IN SAND

Ref. API RP 2A §2.29f

$$p_{us} = A\left\{\frac{\gamma' H}{D}\left[\frac{K_o H \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} + \right.\right.$$

$$\frac{\tan\beta}{\tan(\beta-\phi)}(D + H\tan\beta\tan\alpha) +$$

$$\left.\left. K_o H \tan\beta(\tan\phi \sin\beta - \tan\alpha) - K_a D\right]\right\}$$

$$p_{ud} = A\left[K_a \gamma' H(\tan^8\beta - 1) + K_o \gamma' H \tan\phi \tan^4\beta\right]$$

where

p_u = ultimate resistance, psi
(s = shallow ; d = deep)

A = empirical adjustment factor (Fig. 2.29f. API RP 2A)

γ' = effective sand weight, lb/in³

H = depth, in

K_o = earth pressure @ rest coefficient (0.4)

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ϕ = angle of internal friction of sand, deg.

$\beta = 45° + \phi/2$

$\alpha = \phi/2$

D = pile diameter, in

K_a = Rankine minimum active earth pressure coefficient $(\tan^2(45° - \phi/2))$

For <u>clean sand</u>

$\phi = 35°$ $\qquad \tan\phi = 0.7$

$\beta = 45° + \phi/2 = 62.5$ $\qquad \tan\beta = 1.921$

$\tan^4\beta = 13.617$

$\tan^8\beta = 185.433$

$\sin\beta = 0.887$

$\beta - \phi = 27.5°$ $\qquad \tan(\beta - \phi) = 0.521$

$\alpha = \phi/2 = 17.5°$ $\qquad \tan\alpha = 0.315$

$\cos\alpha = 0.954$

$45° - \phi/2 = 27.5°$ $\qquad K_a = 0.271$

$\gamma' = 65\ \#/cu.ft = 0.0376\ lb/cu.in$

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for $D = 12''$

$$P_{us} = A\left\{\frac{0.0376H}{12}\left[\frac{0.4H \times 0.7 \times 0.887}{0.521 \times 0.954} + \frac{1.921}{0.521}(12 + H \times 1.921 \times 0.315) + 0.4H \times 1.921 \times (0.7 \times 0.887 - 0.315) - 0.271 \times 12\right]\right\}$$

$$= A\left\{\frac{0.0376H}{12}\left[0.5H + 44.246 + 2.231H + 0.235H - 3.252\right]\right\}$$

$$\underline{\underline{P_{us} = 0.00313\ AH\left[2.966H + 40.994\right]}}$$

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$$P_{ud} = A\left[0.271 \times 0.0376H\,(185.433 - 1) + 0.4 \times 0.0376H \times 0.7 \times 13.617\right]$$

$$= A\left[1.879H + 0.143H\right]$$

$$\underline{\underline{P_{ud} = 2.022\,AH}}$$

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Woodpile 12"ϕ nominal

PENETRATION H		H/D	A (STATIC)	P_{us}	P_{ud}
FT	(IN)			psi	psi
0	(0)	0	2.85	0	0
1	(12)	1	2.15	6.18	52.17
2	(24)	2	1.50	12.64	72.79
3	(36)	3	1.08	17.98	78.62
4	(48)	4	0.90	24.79	87.35
5	(60)	5	0.88	36.19	106.76
6	(72)	6	0.88	50.48	128.11
7	(84)	7	0.88	67.13	149.47
8	(96)	8	0.88	86.13	170.82
9	(108)	9	0.88	107.48	192.17
10	(120)	10	0.88	131.19	213.53
15	(180)	15	0.88	285.	320.
20	(240)	20	0.88	498.	427.
25	(300)	25	0.88	769.	534.
30	(360)	30	0.88	1,099.	640.
35	(420)	35	0.88		747.
40	(480)	40	0.88		854.

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PENETRATION	Δh	$(p_u)_{AVE}$	$\Delta Q = (p_u)_{AVE} \cdot \Delta h \cdot D$	$Q_u = \Sigma \Delta Q$	$Q_d = \frac{Q_u}{F.S. (=2.0}$
FT	FT	PSF	Lbs	KIPS	KIPS
0				0	0
	1	445	445		
1				0.455	0.23
	1	1,355	1,355		
2				1.81	0.91
	1	2,205	2,205		
3				4.02	2.01
	1	3,079	3,079		
4				7.09	3.55
	1	4,390	4,390		
5				11.48	5.74
	1	6,240	6,240		
6				17.72	8.86
	1	8,468	8,468		
7				26.19	13.10
	1	11,035	11,035		
8				37.23	18.61
	1	13,940	13,940		
9				51.17	25.58
	1	17,184	17,184		
10				68.35	34.18
	5	29,966	29,966		
15				218.18	109.10
	5	51,264	51,264		
20				474.50	237.25
	5	69,192	69,192		
25				820.46	410.23
	5	84,528	84,528		
30				1,243.10	621.55
	5	99,864	99,864		
35				1,742.42	871.21
	5	115,272	115,272		
40				2,318.78	1,159.39

D.6 Pile Driving Resistance in Sand

Driving resistance of the 12" Ø nominal fender pile is assumed to be the sum of the pile skin friction and the end point bearing capacities.

The results are used to estimate the pile driver (hammer) requirements.

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PILE DRIVING RESISTANCE IN SAND

(Ref. API RP 2A §2.28 8th Edition)

Pile axial bearing capacity

$$Q_u = f A_s + q A_p$$

where

Q_u = ultimate bearing capacity

f = unit skin friction, lbs/ft²

A_s = side surface area of pile, ft²

q = unit end bearing capacity, lbs/ft²

A_s = gross end area of pile, ft²

Sand

$$f = K p_o \tan \phi'$$

where K = coefficient of lateral earth pressure
1.0 for driven pile

p_o = effective overburden pressure, lbs/ft²

ϕ' = angle of soil friction on pile wall
33° for sand on wooden pile

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$q = p_o N_q$

where N_q = bearing capacity factor
40 for clean sand.

for 12"ϕ nominal pile

$$A_p = \frac{\pi}{4} D^2 = \frac{1}{4} \pi \times 1^2 = 0.785 \ \square'$$

$$A_s = \pi D \cdot \Delta h = 3.14(\Delta h) \ \square'$$

$$p_o = \gamma' h$$

γ' = submerged unit weight of sand
65 lbs/cu.ft for clean sand

h = pile penetration

$$f = 1.0 \times 65h \times \tan 33° = 42.21 \cdot h \ \text{lbs/ft}^2$$

$$q = 65h \times 40 = 2{,}600h \ \text{lbs/ft}^2$$

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END BEARING

$q = 2{,}600\,h$
$A_p = 0.785\ \square'$

PENETRATION h	q	$Q_p = q A_p$
FT	LBS/FT²	LBS
0	0	0
1	2,600	2,041
2	5,200	4,082
3	7,800	6,123
4	10,400	8,164
5	13,000	10,205
6	15,600	12,246
7	18,200	14,287
8	20,800	16,328
9	23,400	18,369
10	26,000	20,410
15	39,000	30,615
20	52,000	40,820

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ULTIMATE CAPACITY

PENETRATION	Q_s	Q_p	$Q_u = Q_s + Q_p$
FT	LBS	LBS	LBS
0	0	0	0
1	66	2,041	2,107
2	265	4,082	4,347
3	596	6,123	6,719
4	1,033	8,164	9,197
5	1,633	10,205	11,838
6	2,359	12,246	14,605
7	3,220	14,287	17,507
8	4,214	16,328	20,542
9	5,341	18,369	23,710
10	6,600	20,410	27,010
15	14,884	30,615	45,499
20	26,481	40,820	67,301

D.7 Berthing Velocity Limitation

The berthing velocity perpendicular to the pier is limited by the berthing energy and transferred force computed previously in the Appendix D.4. An additional constraint of the pier loading frame strength is then imposed at the upper limit of the berthing velocity.

The berthing velocity is evaluated for various sizes of tankers. The tonnage described in this section refers to the actual tonnage of the tanker at the time of berthing operation.

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BERTHING VELOCITY LIMITATION

(Refs. 1, 2, 6, 11)

Berthing Energy $E = C_B \frac{W V^2}{2g}$

where $W = W_a + W_b$

W_a = added mass tonnage assumed 60% of actual tonnage (W_b) (Long-tons)

W_b = actual tonnage (Long tons) (tanker tonnage at the time of berthing operation)

$W = 1.6 W_b$

C_B = Berthing Coefficient, 0.5

$g = 32.2$ ft/sec²

V = berthing velocity perpendicular to the dock, ft/min.

$$V = 2\sqrt{\frac{Eg}{W}} = 60 \times 2 \times \sqrt{\frac{32.2 E}{2.24 \times 1.6 W_b}} = 360\sqrt{\frac{E}{W_b}}$$

where E = berthing energy, ft-kips
W_b = actual tonnage, Long-tons

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Water Level @ EL.(+) 0'-0"

Berthing Energy
$E = 42.5$ ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	23.46
12,000	21.42
14,000	19.84
16,000	18.56
18,000	17.50
20,000	16.60
22,000	15.82
24,000	15.14
26,000	14.56
28,000	14.02
30,000	13.54
32,000	13.12
34,000	12.72
36,000	12.36
38,000	12.04
40,000	11.74

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Water Level @ EL.(+) 1'-0"

Berthing Energy
E = 47.0 ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	24.68
12,000	22.52
14,000	20.84
16,000	19.52
18,000	18.40
20,000	17.46
22,000	16.64
24,000	15.94
26,000	15.30
28,000	14.74
30,000	14.24
32,000	13.80
34,000	13.38
36,000	13.00
38,000	12.66
40,000	12.34

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Water Level @ EL.(+)2'-0"

Berthing Energy
$E = 59.0$ ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	27.66
12,000	25.24
14,000	23.38
16,000	21.86
18,000	20.62
20,000	19.56
22,000	18.64
24,000	17.84
26,000	17.14
28,000	16.52
30,000	15.96
32,000	15.46
34,000	15.0
36,000	14.58
38,000	14.18
40,000	13.82

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Water Level @ EL.(+) 3'-0"	Berthing Energy $E = 65.0$ ft-kips
TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	29.02
12,000	26.50
14,000	24.52
16,000	22.94
18,000	21.64
20,000	20.52
22,000	19.56
24,000	18.74
26,000	18.00
28,000	17.34
30,000	16.68
32,000	16.22
34,000	15.74
36,000	15.30
38,000	14.88
40,000	14.52

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Water Level @ EL.(-) 1'-0"   Berthing Energy $E = 37.0$ ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	21.90
12,000	19.98
14,000	18.50
16,000	17.32
18,000	16.32
20,000	15.48
22,000	14.76
24,000	14.14
26,000	13.58
28,000	13.08
30,000	12.64
32,000	12.24
34,000	11.88
36,000	11.54
38,000	11.24
40,000	10.94

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Water Level @ EL.(-)2'-0"   Berthing Energy
$E = 33.0$ ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	20.68
12,000	18.88
14,000	17.48
16,000	16.34
18,000	15.42
20,000	14.62
22,000	13.94
24,000	13.34
26,000	12.82
28,000	12.36
30,000	11.94
32,000	11.56
34,000	11.22
36,000	10.90
38,000	10.60
40,000	10.34

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Water Level @ EL.(-13'-0")   Berthing Energy
E = 26.0 ft-kips

TANKER ACTUAL TONNAGE	LIMITING BERTHING VELOCITY (PERPENDICULAR TO DOCK)
LONG TONS	FT/MIN.
10,000	18.36
12,000	16.76
14,000	15.52
16,000	14.52
18,000	13.68
20,000	12.98
22,000	12.38
24,000	11.84
26,000	11.38
28,000	10.98
30,000	10.60
32,000	10.26
34,000	9.96
36,000	9.68
38,000	9.42
40,000	9.18

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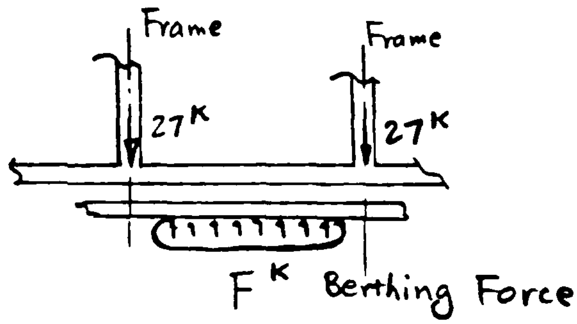
PIER STRENGTH LIMITATION ON LATERAL LOAD

Strength per frame

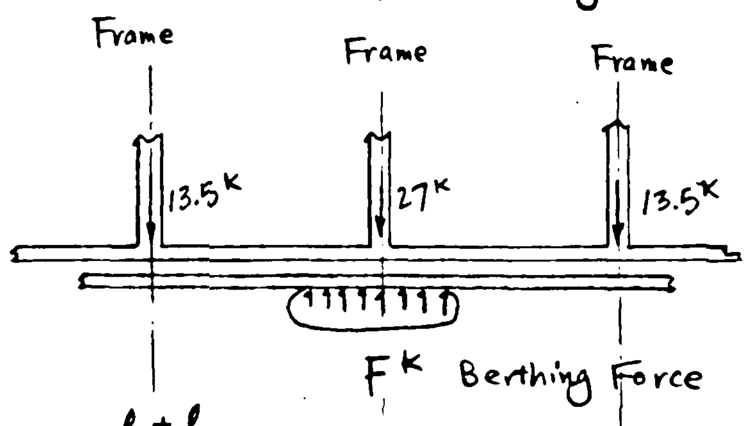
$$P = 27 \text{ kips}$$

Assuming that a combination of two frame strength, i.e., $\bar{P} = 54$ kips, will act to resist berthing force.

<u>Case 1</u>



<u>Case 2</u>



Allowable $F = \dfrac{\ell_1 + \ell_2}{\ell_2} R_t$ $\qquad R_t = \bar{P} = 54^{\text{kips}}$

$$F = \frac{49.5 \times 54}{\ell_2}$$

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when water level @ EL. (+) 3'-0"

$l_2 = 44$

Allowable $F = \frac{49.5 \times 54}{44} = 60.75$ kips < 69.8 kips

4'ϕ x 7.4'l Cushion @ 59% compression

Berthing Energy = 59 ft-kips

D.8 Loading Dolphin Dimensions

Overall dimensions of the friction resistance type dolphins are evaluated herein. The parameters used in the calculations are:

- T-5 tanker at 40,000 DWT
- Tanker approaching velocity perpendicular to the dolphin @ 24 ft/min.
- Dolphin will have

  88 sheet piles type S32 (MP 102)

  Diameter 35'-0"

  Height 90'-0" with 40'-0" penetration in sand

  Rockfill @ 150 #/cu.ft (air weight) above mud-line

  Sand @ 120 #/cu.ft (air weight)
- Hung Fender System

The results of the evaluation are:

- Factor of safety against sliding = 5.0
- Factor of safety against overturning = 2.6

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APPROXIMATE SIZE OF LOADING DOLPHINS

Tanker T-5 Class

W_b, Displacement Tonnage (L.T) = 40,000

D, Draft (ft) = 35

L, Length (ft) = 656

W_a, Added Mass $= \frac{\pi}{4} \rho D^2 L$ (L.T.) = 18,033
($\rho = 64$ #/cu.ft)

W, Total Weight $= W_a + W_b$ (L.T.) = 58,033

V, Berthing Velocity (ft/sec) = 0.4

C_B, Berthing Coefficient = 1.0*

* See NAVFAC DM-26
(Page 26-5-6)

g, Acceleration of Gravity (ft/sec²) = 32.2

$$E = C_B \frac{W V^2}{2g}$$

$$= 1.0 \times \frac{58{,}033 \times (0.4)^2}{2 \times 32.2}$$

$$= 144.2 \text{ ft - L.T.}$$

$$= 323 \text{ ft - kips}$$

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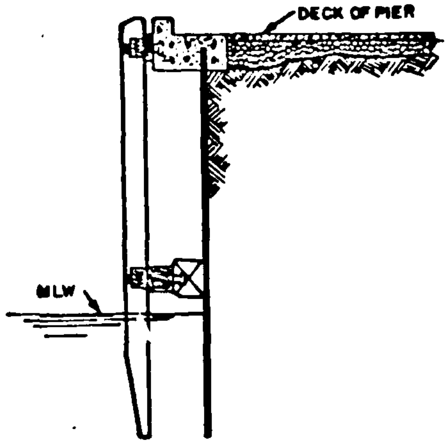

SECTION
(a) FILLED CELLULAR PIER

See NAVFAC DM-25
Pg 25-5-4
Hung Fender System

Assuming that the deflection in the total system, i.e., deflection of the cellular pier, local deflection of the sheet piling, fender cushion deflection and the deflection of ship hull, is 6 inches.

$$F \cdot d = E$$

where F = impact force
d = deflection
E = berthing energy

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$$F = \frac{323}{0.5} = 646 \text{ kips}$$

Assuming that the dolphin will be

88 sheet piles type S32 (MP102)

Diameter 35'-0"

Ht 90'-0"

Rockfill @ 150#/cu.ft (air wt.)

Sand @ 120#/cu.ft (air wt.)

W_d = dolphin dead weight

$$= \frac{\pi}{4} \times 35^2 \times \left[10 \times 150 + 40 \times (150-64) + 40 \times (120-64)\right]$$

$$+ \ 88 \times 10 \times 40 + 88 \times 80 \times (40 - 5.23)$$

$$= 6,908,000 + 279,980$$

$$= 7,187,980 \text{ lbs}$$

R_1 = submerged soil pressure

$$= \frac{1}{2} \times (120-64) \times 40 \times (40 \times 35)$$

$$= 1,568,000 \text{ lbs}$$

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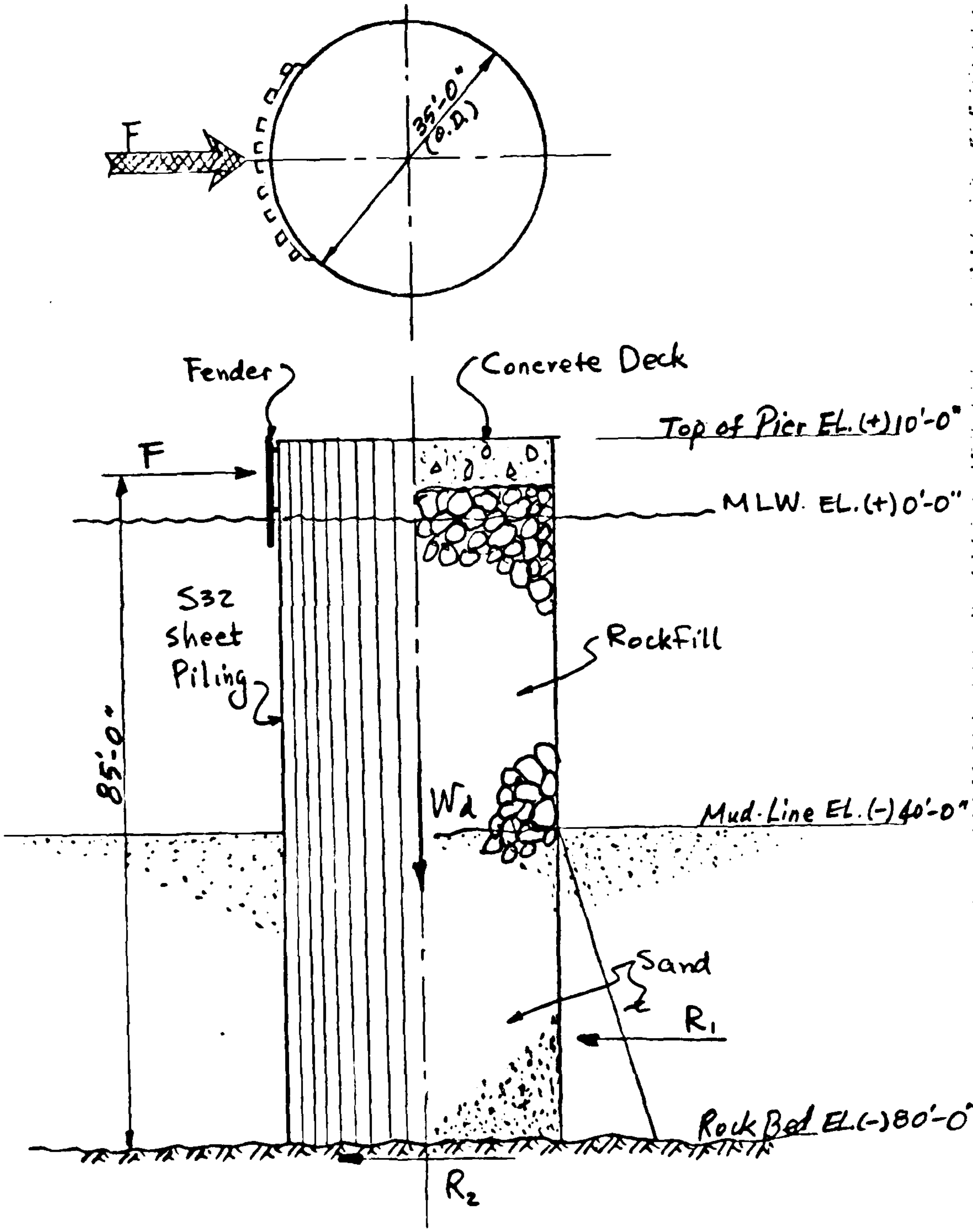

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R_2 = friction force between dolphin base and rock bed

$= f \cdot A$ $\quad$ A = dolphin base area

From API RP 2A §2.27C (Ref. 4)

$f = K\, p_o \tan\phi'$ $\quad$ #/sq. ft

where $K = 0.5$ for axial compressive load

p_o = effective overburden pressure

$= \dfrac{7,187,980}{\frac{\pi}{4} \times 35^2}$

$= 7,471$ #/□'

ϕ' = angle of soil friction on smooth wall

silty sand $\phi' = 25°$

$f = 0.5 \times 7,471 \times \tan 25°$

$= 1,742$ #/SQ.FT

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$$R_2 = 1{,}742 \times \frac{\pi}{4} \times 35^2$$
$$= 1{,}676{,}000 \text{ lbs}$$

Total Horizontal Resistance against sliding

$$R_1 + R_2 = 1{,}568 + 1{,}676$$
$$= 3{,}244 \text{ kips.}$$

F.S. against sliding

$$(F.S.)_{sliding} = \frac{3{,}244}{646} = 5.0$$

$$\text{Overturning Moment} = 646 \times 85$$
$$= 54{,}910 \text{ ft-kips}$$

$$\text{Resisting Moment} = 7{,}188 \times \frac{35}{2} + 1{,}568 \times \frac{40}{3}$$
$$= 141{,}470 \text{ ft-kips}$$

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F.S. against overturning

$$(F.S.)_{O.T.} = \frac{141,470}{54,910} = 2.6$$

END
FILMED
4-86
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